Vibrations in timber bridges due to pedestrian induced forces

A case study of Älvsbackabron

Master of Science Thesis in the Master's Programme Structural Engineering and Building Performance Design

HANNA JANSSON
ISAK SVENSSON

Department of Civil and Environmental Engineering
Division of Structural Engineering
Steel and Timber Structures
CHALMERS UNIVERSITY OF TECHNOLOGY
Göteborg, Sweden 2012
Master's Thesis 2012:96

MASTER’S THESIS 2012:96

Vibrations in timber bridges due to pedestrian induced forces

A case study of Älvsbackabron

Master of Science Thesis in the Master’s Programme Structural Engineering and Building Performance Design

HANNA JANSSON

ISAK SVENSSON

Department of Civil and Environmental Engineering
Division of Structural Engineering
Steel and Timber Structures
CHALMERS UNIVERSITY OF TECHNOLOGY

Göteborg, Sweden 2012

Vibrations in timber bridges due to pedestrian induced forces
A case study of Älvsbackabron

Master of Science Thesis in the Master's Programme Structural Engineering and Building Performance Design

HANNA JANSSON
ISAK SVENSSON

Examensarbete / Institutionen för bygg- och miljöteknik,
Chalmers tekniska högskola 2012:96

Department of Civil and Environmental Engineering
Division of Structural Engineering
Steel and Timber Structures
Chalmers University of Technology
SE-412 96 Göteborg
Sweden
Telephone: + 46 (0)31-772 1000

Cover:
Overview of Älvsbackabron over Skellefteå river in the centre of Skellefteå. More information about Älvsbackabron and Skellefteå can be found in Chapter 6.

Reproservice, Chalmers University of Technology, Göteborg, Sweden 2012

Vibrations in timber bridges due to pedestrian induced forces
A case study of Älvsbackabron

Master of Science Thesis in the Master's Programme Structural Engineering and Building Performance Design

HANNA JANSSON
ISAK SVENSSON
Department of Civil and Environmental Engineering
Division of Structural Engineering

Steel and Timber Structures

Chalmers University of Technology

ABSTRACT

One of the greatest challenges for structural engineers when constructing lightweight timber bridges is the pedestrian induced vibrations. The purpose of this Master's thesis was to investigate the force models, regulations and comfort criteria regarding vibrations in timber footbridges induced by pedestrian forces presented in design codes. Furthermore, a case study of Älvsbackabron, a cable-stayed timber footbridge with span of 130 meters designed by COWI AB, was performed with the purpose of comparing measured accelerations with calculated accelerations and acceleration limit values. Two models were established in the finite element software Brigade/Plus, one with material parameters according to BRO 2004 and the other with material parameters according to Eurocode 5. The results from these two models were compared with each other to study the differences the transition from BRO 2004 to Eurocode implied. Besides, the resulting accelerations from the force models presented in BRO 2004 and the ISO 10137 standard applied on the finite element models were compared with the measured accelerations at Älvsbackabron.

The main differences between the two bridge codes were that lateral vibrations also have to be considered when designing according to Eurocode. When performing dynamic design according to Eurocode 5 the force model presented in the ISO 10137 standard can be used for both vertical and lateral pedestrian forces. In this Master's thesis simplifications of the vertical force model were made which resulted in a practical and useful force model. According to this simplified force model the vertical acceleration limit was fulfilled for a group of twenty pedestrians, which was considered a reasonable design situation.

From the measured accelerations at Älvsbackabron the damping factor was calculated to 1.2% of critical damping which is twice the value used in design. From the field tests at Älvsbackabron it was also concluded that the measured accelerations never exceeded the limit accelerations presented in BRO 2004 and Eurocode 0.

Key words: Älvsbackabron, footbridge, timber bridge, pedestrian induced vibrations, acceleration measurements, damping factor, BRO 2004, Eurocode, ISO 10137

Vibrationer i träbroar orsakade av fotgängare
En fallstudie av Älvsbackabron
Examensarbete inom Structural Engineering and Building Performance Design
HANNA JANSSON, ISAK SVENSSON
Institutionen för bygg- och miljöteknik
Avdelningen för konstruktionsteknik
Stål- och träbyggnad
Chalmers tekniska högskola

SAMMANFATTNING

Under sommaren 2011 färdigställdes Älvsbackabron, en gång- och cykelbro över Skellefteälven i centrala Skellefteå. Älvsbackabron är en 130 meter lång snedstagsbro byggd i trä där en av de största konstruktionstekniska utmaningarna för COWI AB var att bedöma brons dynamiska respons från svängningar orsakade av fotgängare. En del av examensarbetets syfte var att studera accelerationsgränsvärden och lastmodeller i designkoder gällande vibrationer i lätta gång- och cykelbroar. Dessutom genomfördes en fallstudie av Älvsbackabron vilken innefattade både modellering av bron i finita elementprogrammet Brigade/Plus samt accelerationsmätningar i brons gångbana. Målet med fallstudien var att jämföra uppmätta accelerationer från Älvsbackabron med accelerationsgränsvärden samt att beräkna en dämpningsfaktor baserad på de uppmätta accelerationerna.

Som en del av arbetet etablerades två finita element modeller där den ena baserades på materialparametrar från BRO 2004 och den andra på materialparametrar från Eurocode 5. Skillnaderna mellan lastmodellerna från BRO 2004 och ISO 10137 standarden studerades genom att jämföra resultaten från lastsimuleringar i de två olika modellerna. Vidare jämfördes även accelerationer från lastsimuleringar med uppmätta accelerationer från Älvsbackabron med syftet att verifiera beräkningsmodellerna. I undersökningen inkluderades enbart vibrationer i gångbanan från gångtrafik.

En av de största skillnaderna som normbytet från BRO 2004 till Eurocode inneburit är att även laterala accelerationskrav och lastmodeller rörande svängningar numera måste beaktas vid dynamisk design av gång- och cykelbroar. Vid dynamisk design enligt Eurocode 5 kan lastmodellen i ISO 10137 standarden användas, där lastmodeller för både vertikala och laterala krafter finns angivna. I examensarbetet gjordes förenklande antaganden angående den vertikala lastmodellen vilket gav en praktisk och användbar lastmodell. Enligt den förenklade lastmodellen uppfylldes accelerationsgränsvärdena för en grupp om tjugo personer vilket ansågs vara en rimlig dimensioneringssituation.

Från de uppmätta accelerationerna på Älvsbackabron beräknades dämpningsfaktorn till 1,2% av kritisk dämpning vilket är dubbelt så högt som designvärdet. De uppmätta accelerationerna visade också att accelerationskraven i BRO 2004 och Eurocode 0 inte överskreds. Detta innebar att Älvsbackabron ansågs uppfylla funktionskravet om att bron ska vara fri från besvärande svängningar.

Nyckelord: Älvsbackabron, gång- och cykelbro, träbro, svängningar orsakade av fotgängare, accelerationsmätning, dämpning, BRO 2004, Eurocode, ISO 10137

Contents

Preface

The thesis is the final part of the civil engineering programme and has been carried out from January to June 2012 at COWI AB, Göteborg, in cooperation with the Division of Structural Engineering at Chalmers University of Technology, Sweden. Professor Robert Kliger from the Division of Structural Engineering was the examiner for the thesis.

In this Master’s thesis a study of pedestrian induced vibrations in lightweight timber bridges have been carried out with Älvsbackabron in Skellefteå as a case study. Comprised within the case study are two testing occasions at Älvsbackabron where the measuring equipment was borrowed from the Division of Dynamics and the Division of Structural Engineering at Chalmers University of Technology, Sweden. For this we are very grateful and without the equipment a large part of the thesis could not have been accomplished.

We would like to thank our supervisors Robert Kliger and Tomas Svensson who have motivated and supported us during our work and contributed with valuable knowledge and experience. We would also like to thank Thomas Hallgren at COWI for all his help during our modelling process and Peter Jacobsson at Martinsons Träbroar who organised the two testing occasions at Älvsbackabron. We appreciate all help from those at Martinsons Träbroar, COWI, SP Trätek and Luleå University of Technology that participated in our tests and made our trips to Skellefteå memorable and successful.

The computer modelling of Älvsbackabron is performed in Bridgade/Plus, a finite element software specialised for bridge design. A special thanks to Scanscot and their contribution by providing us a licence and valuable support. We are also very grateful for the scholarships we received from the funds Chalmers MasterCard and Chalmers Vänner. Their contribution helped us finance our visits to Skellefteå.

During our work we have been encouraged, motivated and inspired from the positive work environment at COWI, why we want to thank all the co-workers at COWI.

Finally, we would like to thank our opponents David Glans and Fredrik Eckerwall for a great cooperation throughout the project.

Göteborg June 2012

Hanna Jansson

Isak Svensson

Notations

Roman upper case letters

A	load amplification factor
B	width of the bridge [m]
$\mathbf{C}$	structural damping matrix
$C(N)$	coordination factor
F	pedestrian force [N]
$F(t)$	pedestrian force [N]
$F(t)_N$	total force from N pedestrians [N]
G	static load from pedestrian [N]
$G(f_B)$	pedestrians frequency synchronization coefficient
$H(\dot{u}_B)$	pedestrian girder movement synchronization coefficient
$\mathbf{K}$	structural stiffness matrix
L	length of the bridge [m]
$\mathbf{M}$	structural mass matrix
$M_P g$	modal self-weight of the pedestrian
N	number of people in the group
P	amount of pedestrians
Q	static load from pedestrian [N]
T	period of a step

Roman lower case letters

a_i	acceleration at cycle i [m/s^2]
a_{i+1}	acceleration at cycle i+1 [m/s^2]
a_{peak}	peak acceleration limit [m/s^2]
a_v	acceleration limit in vertical direction [m/s^2]
d	density of pedestrian traffic
f	frequency of the loading [Hz]
f_0	natural frequency of the bridge [Hz]
f_F	natural frequency of the bridge [Hz]
$\boldsymbol{f}(t)$	external force
g	gravity constant [m/s^2]
h	total number of pedestrians
i	number of steps

j	is the pedestrian number
k	number of harmonics of interest
k_0	factor expressing contact time
k_1	bridge size coefficient
k_2	frequency coefficient
k_3	numerical coefficient
k_4	lateral force / pedestrian weight ratio
k_5	girder vibration synchronization coefficient
k_6	bridge dependent numerical coefficient
m	mass of the runner [kg]
n	is the integer number of the natural harmonic
t	time [s]
$\boldsymbol{u}(t)$	displacement of the structure
$\dot{u}$	lateral velocity of the bridge
$\dot{u}_B$	velocity of the bridge
$\dot{\boldsymbol{u}}(t)$	velocity of the structure
$\ddot{\boldsymbol{u}}(t)$	acceleration of the structure

Greek letters

α_i	coefficient corresponding to the i:th harmonic
α_n	numerical coefficient
φ_i	phase lag for the i:th harmonic
ϕ_i	mode shape
ϕ_n	phase angle for the n:th harmonic
ζ	damping factor

1 Introduction

In the late summer of 2011 a cable-stayed timber footbridge was completed in the centre of Skellefteå. The bridge is called Älvsbackabron and has a free span of 130 meter which makes it the largest cable-stayed timber bridge in Scandinavia today. The client was the municipality of Skellefteå and Martinsons Träbroar AB acted as the contractor with COWI AB as the structural engineering consultant (Byggindustrin, 2010).

Älvsbackabron is a part of a research program called Smart Wooden Bridge in Smart City which aims to increase the knowledge of advanced timber bridges and consequently strengthen the timber industry in Sweden. During the erection of the bridge measuring devices were installed and from these devices data are collected and analysed. The intention of collecting the data is to develop models and tools for estimating the technical performance and quality of timber bridges. The municipality of Skellefteå, Luleå University of Technology, SP Trätek and Martinsons Träbroar are the participants of the project which is partly financed by the European Regional Development Fund (Degerfeldt, 2009).

Timber footbridges with span greater than 30 meters are sensitive to vibrations which is why dynamic considerations are needed in the design (Pousette, 2001). A source for vibrations is pedestrian traffic and a special loading situation is caused by a crowd of pedestrians walking with the same pace on the bridge. This loading situation was one of the greatest challenges when designing Älvsbackabron together with predicting how the dynamic behaviour of the modelled bridge would correlate to the real response of the bridge (Martinsons, 2009).

The general design rules in Eurocode 5 regarding vibrations in timber footbridges state that a footbridge should be designed in a way so that the loads on the bridge don not result in uncomfortable vibrations for the users (Anon. 2004b). However, the question of how to define uncomfortable vibrations remains. The experience of vibrations is highly individual why it is difficult to specify suitable regulations for vibrations in lightweight bridges (Pousette, 2001). In Eurocode 0 (Anon., 2002b) acceleration limits regarding pedestrian induced vibrations are stated, but no methods for assessing the dynamic behaviour are given. Instead, it is up to the designer to make reasonable assumptions which ensures that the limits are fulfilled.

One assumption made in the design process that affects the accelerations is the damping factor. In the dynamic design of Älvsbackabron, by COWI, the damping factor from BRO 2004 is used. As a consequence the calculated design values of the accelerations in the bridge deck exceed the limit values. However, according to Thomas Hallgren[1] the dynamic designer of Älvsbackabron, the damping factor is assumed to be twice as high as the value given in BRO 2004. This assumption is based on higher values of the damping factor presented in literature, such as Eurocode 5 (Anon., 2004b). A damping factor twice as high as the value in BRO 2004 results in lower accelerations and fulfils the design limits. However, if the acceleration limits would not be fulfilled preparations to install dampers are made on the bridge.

Älvsbackabron is designed according to the Swedish Road Administration Bridge Code BRO 2004, but since 2009 Eurocode has to be used in all designs. The effects

[1] Thomas Hallgren, structural engineer COWI AB, meeting March 21:th 2012

on the dynamic design from this transition are studied to relate Eurocode to BRO 2004 and get useful experience for future projects.

1.1 Purpose

The aim of the Master´s thesis is to investigate the dynamic force models, regulations and comfort criteria regarding pedestrian induced vibrations in the serviceability limit state for lightweight timber bridges given in BRO 2004, Eurocode and the international standard ISO 10137. The purpose of the investigation is to find similarities and differences between the dynamic force models, regulations and comfort criteria presented in the studied codes and standard.

In the Master's thesis, a case study of Älvsbackabron is also included. The purpose of the case study is to compare the resulting accelerations from the force models presented in BRO 2004 and Eurocode. Furthermore, the effects of the transition from BRO 2004 to Eurocode on the dynamic design of timber footbridges with respect to pedestrian induced vibrations are studied.

An additional purpose of the case study is to measure the vertical and lateral accelerations of Älvsbackabron to evaluate if the regulations regarding accelerations in Eurocode and BRO 2004 are fulfilled. Besides, a damping factor of the bridge is estimated from the results of the measurements.

1.2 Limitations

Besides the dynamic forces from pedestrian loading, wind loads may also excite bridge structures and cause disturbing sway and motion why an assessment of the aerodynamic behaviour of new bridges is of great importance in design. However, this study is limited to the dynamic forces induced by pedestrians and the corresponding bridge accelerations. Moreover, the contribution to the dynamic force from bicycle traffic is neglected since the force does not vary in amplitude as the force from pedestrian traffic.

Dynamic design is needed for slender, lightweight cable-stayed footbridges with free span above 30 meters, why this study is limited to footbridges with span length above this limit (Pousette, 2001). As the span length increases the natural frequency of a bridge decreases resulting in a bridge which can be more susceptible to vibrations (Stoyanoff & Hunter, 2003). 200 meters have been set as an upper limit for the span length.

The codes regarding dynamic design of timber bridges studied in detail in this thesis are BRO 2004, Eurocode and the international standard ISO 10137. There is no force model for pedestrian induced vibrations in Eurocode, instead the designer is referred to the national annex or project specific models. However, no such model is given in the Swedish national annex, why this study of force models in Eurocode is restricted to the complementary ISO 10137 standard. In the case study only the resulting accelerations in the midspan from the force models presented in BRO 2004 and the ISO 10137 standard are compared.

To evaluate the dynamic behaviour of Älvsbackabron vertical and lateral accelerations are measured with accelerometers attached to the bridge deck during the tests. According to Craig and Kurdila (2006) accelerations is the most common quantity to measure when estimating the dynamic response of a structure, why accelerometers are used as measuring device. Furthermore, the accelerations of the pylons are not measured at the test occasions.

1.3 Method

The first part of the report is a literature study of the dynamic behaviour of lightweight cable-stayed footbridges including regulations and recommendations for vibrations and dynamic force models for pedestrian induced forces. The studied regulations, comfort criteria and force models are compared respectively with the intention to find similarities and differences. In addition, a section describing different dynamic loading tests is included. This part of the report is mainly based on the chapters regarding dynamic design of footbridges in BRO 2004, ISO 10137, Eurocode 0, Eurocode 1 and Eurocode 5. Besides these codes, some regulations and force models presented in research literature are studied with the intention to expand the comparison. This first part intends to introduce the reader to the subject and give the needed theory for assessing the dynamic response of Älvsbackabron.

Following the literature study the second part of the report is presented, a case study of the dynamic behaviour of Älvsbackabron. As an introduction, the city of Skellefteå with the timber processing industry and the research project Smart Wooden Bridge in Smart City are described. Moreover, descriptions of both the actual and the modelled dynamic behaviour of Älvsbackabron are presented.

In this thesis, Älvsbackabron is modelled with the finite element software Brigade/Plus, specialised for modelling bridge structures. Two models of Älvsbackabron are established in this Master's thesis, one with material parameters according to BRO 2004 and the other with material parameters according to Eurocode 5. In the report, these models are referred to as Master's thesis model according to BRO 2004 and Master's thesis model according to Eurocode. The Master's thesis model according to BRO 2004 is verified with the dynamic design model by COWI. The verification is described in detail in Section 7.2. The Master's thesis models are used to compare the resulting accelerations from the force models given in BRO 2004 and ISO 10137.

In connection to the case study two test occasions and loading simulations are described, for a more detailed description, see Section 8.2 and Section 8.3. The data from the tests are analysed, processed and visualised using the programming software MATLAB. Two of the dynamic loading tests at Älvsbackabron are also simulated in the Master's thesis model according to Eurocode and the resulting accelerations are used in the comparison part. The case study is based on the results from the models and the measurements from the field tests at the bridge.

Finally, comparisons between the results from the literature studie and the case study are presented. From the literature part comparisons between the described regulations, comfort criteria and pedestrian force models are made respectively. In the case study comparisons of the results from the two Master's thesis models are made. The models are compared with respect to material parameters, natural frequencies and resulting accelerations from the force models in BRO 2004 and ISO 10137.

2 Vibrations in lightweight cable-stayed bridges

The aspiration of using sustainable building materials in combination with increased knowledge in advanced technology produces new opportunities in the design of new structures. When it comes to footbridges more spectacular designs can be accomplished, for example cable-stayed bridges with large span length (Melchor Blanco et al., 2005). Large spans in combination with a lightweight material such as steel or timber gives a slender bridge structure with low mass inertia which in turn results in low natural frequencies of the bridge. Forces caused by pedestrians walking on the bridge can result in loading frequencies within the same range as the natural frequencies of the bridge. If the walking frequencies coincide with the natural frequencies of the bridge resonance can occur why dynamic design of these types of bridges is necessary (Heinemeyer et al., 2009).

In this chapter the origin of the dynamic forces induced by pedestrians in lightweight bridges is described and how they can cause vibrations in a bridge. In addition, a description of human reaction to different types of vibrations is included and a mathematic model for vibrations is described.

2.1 Vibrations induced by pedestrians

The dynamic forces acting on a bridge deck origin from walking, running and jumping pedestrians. The contribution to the dynamic force from bicycle traffic can be neglected since the force does not vary in amplitude as the force from pedestrian traffic. The pedestrian induced live loads can excite the bridge and cause vibrations in both vertical and lateral direction depending on the coincidence of the frequencies. However, there is little documentation of pedestrian induced vibrations causing collapse or damage of a bridge in the ultimate state. Instead, the dynamic vibration problem is a serviceability problem why a dynamic analysis is sufficient in the design phase to assure that the bridge is free from disturbing sway (Heinemeyer et al., 2009).

2.1.1 Dynamic forces from pedestrians

When a pedestrian walks with a walking frequency of 2 Hz, up to 40% of the self-weight is transmitted as dynamic vertical force acting on the bridge deck (Dallard et al., 2001a). Besides the vertical component, the walking mechanism also creates two horizontal components, one in lateral direction across the bridge and one longitudinal in the direction of the traffic (Bachmann & Ammann, 1987). These forces are shown in Figure 1.

The horizontal forces are smaller than the vertical force, as an example the lateral component is ten times smaller than the dynamic vertical force. Especially the lateral force can cause problems with sway in the bridge deck where the most famous case is the Millennium Bridge in London (Dallard et al., 2001a). The longitudinal component can in rare cases cause problem with disturbing motion for a bridge with low stiffness in longitudinal direction (Bachmann & Ammann, 1987).

Figure 1 Directions of the vertical, lateral and longitudinal component of the dynamic force from a pedestrian.

Bachmann and Ammann (1987) assign the dynamic load induced by a walking or running pedestrian as a periodic load meaning that the load value varies in time, but the variation is repeated over a certain time interval. A footbridge can also be exposed to transient loading where the load varies in time without periodicity. An example is landing from a jump which results in transient loading.

In general, the normal walking frequency for a pedestrian is about two steps per second, 2 Hz, which results in an approximate forward speed of 1.5 m/s with a stride length of 0.75 m. An increasing pace rate results in a vertical dynamic load with increased magnitude plus a shortening of the contact period between the foot and the ground (Bachmann & Ammann, 1987). This relationship is illustrated in the Figures 2-4.

Figure 2 shows a footfall for a pedestrian with a walking frequency of 1.67 Hz. It can be seen that the maximum value of the vertical load is almost the same as the self-weight. The two peak values of the curve represent the impact from the heel and the forefoot respectively (Bachmann & Ammann, 1987).

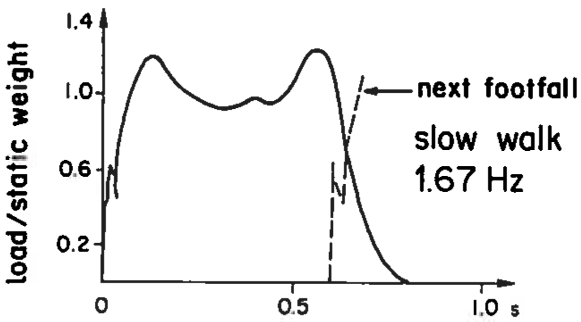


Figure 2 *Dynamic vertical load versus the time for one footfall at walking frequency 1.67 Hz (Bachmann & Ammann, 1987).*

A higher walking frequency results in increased maximum amplitude of the dynamic vertical force, illustrated in Figure 3 where a footfall with walking frequency of 2.38 Hz is shown. Notable is also the shortening of the contact time for the foot compared to the previous figure (Bachmann & Ammann, 1987).

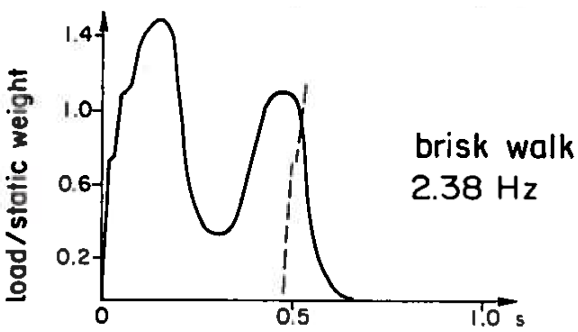


Figure 3 *Dynamic vertical load versus the time for one footfall at walking frequency 2.38 Hz (Bachmann & Ammann, 1987).*

A jogging or running pedestrian exerts a higher dynamic vertical force than the two previous cases. The load-time curve becomes smoother since the contact time for one foot is shorter (Bachmann & Ammann, 1987). A footfall for a jogging pedestrian is shown in Figure 4.

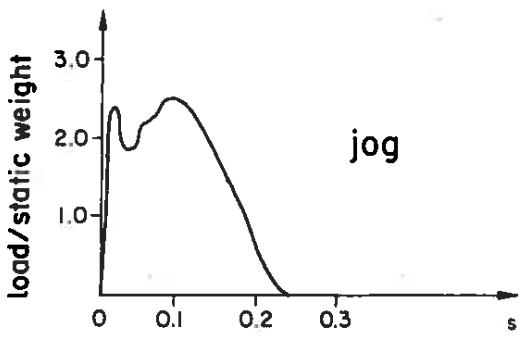


Figure 4 *Dynamic vertical load versus the time for one footfall when jogging (Bachmann & Ammann, 1987).*

An aspect influencing the walking frequency is the amount of pedestrians crossing the bridge at the same time. If the bridge is heavily crowded each individual cannot move freely, instead the crowd adapts its walking speed which is lowered. Consequently, the walking frequency a bridge is subjected to depends both on each pedestrian and on the amount of people crossing the bridge at the time. The expected walking frequency in vertical direction is in the region 1.2-2.2 Hz (Dallard et al., 2001a).

The vertical vibrations are according to Dallard et al. (2001a) investigated in research to a greater extent than the lateral vibrations and there are some codes regarding the assessment and design concerning vertical vibrations. However, the lateral forces causing the bridge to sway in lateral direction are not as documented, but can still cause problems with uncomfortable vibrations.

2.1.2 Dynamic lateral forces

When a person walks the gravity centre is shifted from right to left repeatedly as the person steps with the right and left foot. This motion is causing the lateral force and the frequency of this force is in the range of 0.6-1.1 Hz, which is half the frequency of the vertical load. The reason is that only every second step gives rise to a lateral force in each direction. When a person steps with the right foot a force directed to the right is created. In the next step the person steps with the left foot and consequently creating a force directed to the left (Dallard et al., 2001a). Figure 5 illustrates how the gravity centre is shifted for a pedestrian and how the lateral forces are created from alternate steps.

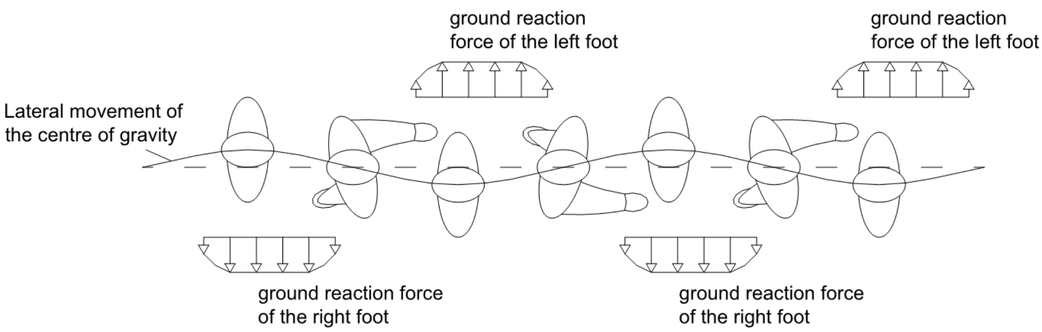


Figure 5 Lateral forces caused by the lateral movement of the gravity centre of a pedestrian (Heinemeyer et al., 2009).

If some frequencies within the frequency range of the lateral force are close to the natural frequencies of the lightweight bridge resonance can occur. One person alone cannot cause any significant lateral vibration in a bridge, but if a crowd walks synchronised lateral vibrations are possible. Kawasaki and Nakamura (2006) describes a test at a bridge where 30 students sidestepped at the same time with a frequency close to the natural frequency of the bridge causing noticeable vibrations.

Furthermore, if the lateral forces from pedestrians do not have the same phase, the phases will cancel each other out and no resonance occur. Kawasaki and Nakamura (2006) explains the resonance phenomena as the human intuition of synchronizing with the vibrations. When the bridge starts to sway a pedestrian tends to move in the same direction as the bridge, e.g. when the bridge sway to the right a pedestrian steps with the right foot. This results in increased vibration amplitude in the bridge, resonance. In some literature this phenomena is also called lock-in (Dallard et al., 2001a). However, with this theory the vibrations would increase to infinity which is not the case in bridges where problems with lateral vibrations are observed. Kawasaki and Nakamura (2006) explain this with the human ability of adapting the walking speed when the magnitude of the vibrations becomes too large. When the oscillations of the bridge grow too large from the synchronized walking pace, the pedestrians tend to lower their walking speed or grab the hand rail, which wearies out the increasing amplitude.

It is emphasised by Dallard et al. (2001a) that further measurements and research is needed to establish the magnitude of the lateral force and its effect on the motion in the bridge and at which magnitude of the sway lock-in occurs. Moreover, Ingólfsson and Georgakis (2011) means that the importance of the pedestrian synchronization with the bridge motion is not clearly verified in tests and the synchronization may not be a decisive condition for lateral vibrations to occur.

2.2 Human perception of bridge vibrations

Vibrations in bridges are normally perceived as uncomfortable by the users, but at which level vibrations become disturbing is not only highly individual but one person´s perception can vary from one day to another. To assess the right serviceability class for a bridge it is important to identify the tolerance level and recommendations for vibrations in bridges (Živanović et al., 2005).

The human body can sense vibrations from below 1 Hz up to 100 kHz and there are several ways people tend to feel pedestrian induced vibrations. For example, vibrations in a footbridge can be experienced mechanically, meaning that a person is sensing the deck vibrating while crossing the bridge. Vibrations could also be experienced visually by noticing sway of the bridge deck or movement in the cables (Bachmann & Ammann, 1987).

According to Živanović et al. (2005), it has been shown that humans are more sensitive to vibrations in the lateral direction, but beyond the direction of the vibration there are several other factors influencing the human sensitivity to vibrations. One factor is the body posture, a person sitting or standing still on a bridge is more sensitive to vibrations than a walking person.

Another aspect mentioned by Heinemeyer et al. (2009) is the number of people walking on the bridge at the same time. It has been noted that a higher number of people walking on a bridge gives a higher individual tolerance towards motion and vibrations. The reasons for this, however, have not yet been established. Aspects as exposure time at the bridge, frequency of use and height above ground also influence the individual sensitivity to bridge vibrations (Heinemeyer et al., 2009).

Heinemeyer et al. (2009) present a survey of two footbridges with similar dynamic properties, but with different appearances and locations. The first bridge has a slender look and is located in the countryside while the other is located in a town with a more strong-looking appearance. The results from the survey showed that more people were disturbed by vibrations in the second bridge and the authors mean that this indicates that both appearance and expected stability affects the experience of the bridge (Heinemeyer et al., 2009).

According to Heinemeyer et al. (2009) the matter of pedestrians synchronizing with vertical vibrations is not observed in research. The reason is that the vertical force is absorbed by the pedestrian’s legs and joints hence damping the force resulting in no shift of the gravity centre.

The question of what is a tolerable level of vibration does not have an unambiguous answer. In a literature review by Živanović et al. (2005) different values for serviceability accelerations for both vertical and lateral accelerations are given. To be able to compare the different values presented in the literature all values are converted into peak accelerations.

Mentioned in the review is a research which aims to describe the vertical limit accelerations by taking into account peoples different perceptions of acceleration. Four perception levels were defined varying from barely noticeable to greatly unpleasant with corresponding probability. Results from this method are given as an example for a bridge with a first natural frequency of 2 Hz. The risk that accelerations up to a level of 0.18 m/s^2 are experienced unpleasant is small and a suggested serviceability limit for the vertical accelerations is 0.13 m/s^2 (Živanović et al., 2005). These values are low compared to a constant limit of 0.5 m/s^2 which is recommended by Bachmann and Ammann (1987).

According to Živanović et al. (2005) 1.35 m/s^2 is suggested as a serviceability level for lateral accelerations. The research by Nakamura resulting in this limit is based on people's perception of bridge vibrations from a full scale test at a footbridge. With lateral accelerations of 1.35 m/s^2 the pedestrians felt insecure and their walking patterns were disturbed. Besides the serviceability limit, the value 0.3 m/s^2 is given as a level of acceleration for which it is unlikely that the accelerations are perceived uncomfortable by the users. Limitations regarding both vertical and lateral accelerations are described to a greater extent in Chapter 3.

2.3 Dynamic model for vibrations

To assess vibrations the transmission path, vibration source and receiver of the vibrations need to be determined, which is the first step for characterising structures subjected to disturbing vibrations (Anon., 2007).

For the case of a cable-stayed timber bridge both the vibration source and receiver were described in the previous sections, namely the pedestrians. The transmission path, however, is also important in the matter of preventing uncomfortable motion. For a cable-stayed bridge, it is the bridge deck that acts as the transmission path by transferring vibrations caused by one pedestrian to the receiver, another pedestrian. The characteristics of the transmission path, such as geometric properties, material damping and natural frequencies of the deck influence the amplitude and spreading of the vibrations (Anon., 2007).

The mathematic model describing the dynamic behaviour of a structure with its characteristics of the transmission path and the pedestrian induced force is called the equation of motion and can be seen in Equation (1). It is not only the walking frequency and number of pedestrians that are decisive in the assessment of the dynamic behaviour, but also the mass, stiffness and damping of the bridge play an important role (Dallard et al., 2001a).

$$\mathbf{M}\ddot{\boldsymbol{u}}(\mathrm{t}) + \mathbf{C}\dot{\boldsymbol{u}}(\mathrm{t}) + \mathbf{K}\boldsymbol{u}(\mathrm{t}) = \boldsymbol{f}(\mathrm{t}) \tag{1}$$

where

$\mathbf{M}$ is the structural mass matrix

$\mathbf{C}$ is the structural damping matrix

$\mathbf{K}$	is the structural stiffness matrix
$\boldsymbol{f}(t)$	is the external force
$\ddot{\boldsymbol{u}}(t)$	is the acceleration of the structure
$\dot{\boldsymbol{u}}(t)$	is the velocity of the structure
$\boldsymbol{u}(t)$	is the displacement of the structure

A vibrating system has a stable equilibrium state to which the system tries to return. The force that regains the equilibrium state is called the stiffness force and it is proportional to the displacement of the system. The system must also contain mass to vibrate, which is proportional to the acceleration. Damping is dissipation of energy in a structure meaning that if no external forces are applied the vibrations will fade away with time (Maguire & Wyatt, 2002).

The mass and stiffness of a footbridge can be determined with a finite element model in which the structure is divided into a finite number of elements where the geometric and material properties are assembled into mass and stiffness matrices. The damping matrix, however, is difficult to model mathematically and is most often determined in an experimental way (Živanović et al., 2005).

It can be desirable in design to influence the natural frequencies for a cable-stayed footbridge. This can be accomplished by modifying the geometry or material properties of the structure. By increasing stiffness of the deck, the pylons or increasing the sectional area of the cables, the resulting natural frequency will be higher. Pousette (2001) assigns the sectional area of the cables as the most significant factor for increasing the natural frequency of the bridge. To increase the damping of a structure, dampers can be installed.

According to Živanović et al. (2006), due to uncertainties regarding material properties, support conditions and the effect of non-structural elements, the natural frequencies calculated with a FE model may deviate from the real frequencies of a built bridge. Dynamic testing followed by tuning the FE model is suggested as a working method that should be used to a wider extent in civil engineering to increase the reliability of the models (Živanović et al., 2006).

3 Regulations of vibrations in footbridges

To reduce the risk of pedestrians feeling discomfort or unsafe due to vibrations when walking on a footbridge, regulations regarding vibrations are given in the design codes. The value, sometimes called comfort criteria, indicate up to which limit most people do not feel disturbed by the vibrations of the bridge. The most common factors to set as limit values are either the natural frequency or the acceleration. The acceleration is usually expressed in terms of maximum allowed acceleration in the bridge deck.

It is stressed that there are many uncertainties involved in the assessment of the dynamic performance of a footbridge, e.g. the amount of pedestrians on the bridge during the normal use. This means that if the comfort criteria are not fulfilled with margin it could be necessary to make extra investigations to evaluate if dampers are needed (Anon., 2002b).

In this chapter the comfort criteria regarding pedestrian induced vibrations in footbridges are presented for some codes. Focus in the comparison will be on the comfort criteria given in BRO 2004 and Eurocode 0, but a few additional codes are presented for comparison. The values are compared with the intention to investigate the differences and similarities between different codes.

3.1 Regulations according to BRO 2004

BRO 2004 was published by the Swedish Road Administration and was until 2009 the Swedish standard for bridge design. The bridge code was replaced by Eurocode, but for projects procured before 2009 it is still allowed to use BRO 2004 as design code (Trafikverket, 2012). The structural design of Älvsbackabron was procured before 2009 hence BRO 2004 was the valid design code.

The limit for vertical vibrations in footbridges is expressed in terms of a natural frequency limit. A footbridge should be designed so that the first vertical natural frequency of the bridge deck is higher than 3.5 Hz. During the natural frequency analysis the loads on the bridge should be the self-weight of the structural and non-structural elements and, if applicable, the tension forces (Anon., 2004d).

If the fist natural frequency is below 3.5 Hz, a maximum limit value for the vertical acceleration of the bridge deck is set. This limit value is 0.5 m/s^2 which is the root mean square value of the acceleration. To transform the root mean square value into a peak value it is multiplied by $\sqrt{2}$ which gives a peak limit value for vertical accelerations of approximately 0.7 m/s^2. No requirements or recommendations for lateral vibrations are given in BRO 2004 (Anon., 2004d).

3.2 Regulations according to Eurocode

The Eurocodes are European standards with design rules regarding structural design published by the European Committee for Standardization. In design of timber bridges there are mainly Eurocode 0: Basis of Structural Design, Eurocode 1: Actions on structures and Eurocode 5: Design of timber structures that are used (Anon., 2005).

The general restrictions of design regarding vibrations presented in Eurocode 5: Design of Timber Structures state that the expected actions on the bridge should be controlled so that they do not cause vibrations that may harm the structural function or cause discomfort to the user (Anon., 2004b). In addition, comfort criteria based on the maximum acceleration of the bridge deck should be used in the design. The criteria could be stated in the national annex or set by the client for a specific project. However, in the code there are some recommended values for acceleration limits, see Table 1.

Table 1 Recommended acceleration limits in Eurocode 0 (Anon., 2005).

Acceleration limit [m/s^2]	Direction and occurrence
0.7	Vertical acceleration
0.2	Lateral acceleration, normal use
0.4	Lateral acceleration, exceptional crowd conditions

In dynamic design of footbridges, the natural frequencies are calculated to ensure that the comfort criteria are met. If the natural frequency of the bridge deck is less than 5 Hz for the first vertical mode and less than 2.5 Hz for the first lateral mode a verification of the comfort criteria should be performed (Anon., 2005).

The comfort criteria should also be verified with respect to the vibrations caused by the dynamic pedestrian force. If the frequency of the pedestrian force coincide with one or some natural frequencies of the bridge deck resonance could occur. This effect must be considered in design and if there is a risk for resonance the resulting accelerations must be investigated and fulfil the comfort criteria (Anon., 2003). The forces from a walking and jogging pedestrian can be assumed to have frequencies according to Table 2.

Table 2 Frequencies for normal walking and jogging (Anon., 2003).

Frequency [Hz]	Direction and activity
1-3	Vertical direction, normal walking
0.5-1.5	Lateral direction, normal walking
3	Vertical direction, jogging

In the design of footbridges, the expected pedestrian traffic and design situation should be assessed for each individual bridge. The design situation should be based on the expected traffic during the service life of the bridge. During the service life the bridge is subjected to a load level which can be considered its persistent loading level. Depending on each individual project, occasionally higher load levels must be considered such as choreographic and festive events. The surrounding activities may also influence the amount of traffic on the bridge, for example schools and railway stations. Recommended amount of pedestrians for the persistent load level depends on the considered area of the bridge deck but should be between eight and fifteen people. For the higher load levels, a significantly higher amount of people need to be considered in the design. Still, at present, no verification rule for higher load levels are formulated, but the designer is directed to literature and special studies of the individual case (Anon., 2005).

3.2.1 National annexes to Eurocode

Every country using the Eurocodes has their own national annex which should be used in conjunction with Eurocode. The national annex contains country specific values to the notes in Eurocode which states that the national annex may apply (Anon., 2011).

The Swedish national annex (Anon., 2011) for bridges is issued by the Swedish Transport Administration and contains the national parameters to Eurocode. In the design of footbridges Eurocode 0 states that the national parameters might be applied instead of the comfort criteria suggested in code. In the Swedish annex, no further restrictions regarding the comfort criteria are given. Instead, it is stated that the client is allowed to assign project specific values for each individual project (Anon., 2011).

3.2.2 Suggested design requirements by JRC and ECCC

The Joint Research Centre (JRC) of the European Commission has in collaboration with the European Convention for Constructional Steelworks (ECCC) published a report based on research aiming to implement, develop and improve the Eurocodes regarding design of lightweight footbridges subjected to human induced vibrations. The results presented in the report are supposed to be applicable to Eurocode 0, Eurocode 1, Eurocode 3 and to some parts in Eurocode 5 (Heinemeyer et al., 2009).

To assess suitable design requirements for a bridge, the designer is recommended to classify the usage of the bridge with different traffic situations and comfort classes into several design situations. The design situations should consider different loading situations during the design life of the footbridge and should be individual for each project. An example illustrating different design situations with corresponding traffic and comfort classes is presented in Table 3. A design situation that only occurs once in the design life of a structure is supposed to have a less strict limit regarding accelerations than a design situation occurring daily (Heinemeyer et al., 2009).

Table 3 Example of how the traffic and comfort classes should be used when assigning design situations for footbridges (Heinemeyer et al., 2009).

Design situation	Description	Traffic class	Occurrence	Comfort class
1	Opening	TC 4	Once a lifetime	CL 3
2	Commuters	TC 2	Daily	CL 1
3	Races	TC 3	Once a year	CL 2

It is suggested that the pedestrian traffic should be divided into five levels where the first level, traffic class 1, has very weak pedestrian traffic while traffic class 5 has exceptionally dense traffic (Heinemeyer et al., 2009). The traffic classes are described in Table 4. The density of pedestrian traffic is calculated using Equation (2).

Table 4 Suggested traffic classes (Heinemeyer et al., 2009).

Traffic class	Density, d [P/m²]	Description	Characteristics
TC 1	< 0.2	Very weak traffic	Free crossing, comfortable and free walking, choose own pace
TC 2	0.2	Weak traffic	Free walking, pedestrians can choose their own pacing rate, overtaking possible
TC 3	0.5	Dense traffic	Free walking, overtaking might be restricted
TC 4	1.0	Very dense traffic	Restricted waking, overtaking not possible
TC 5	1.5	Exceptionally dense traffic	Unpleasant waking, cannot choose pace

$$d = \frac{P}{BL} \tag{2}$$

where

d is the density of pedestrian traffic

P is the amount of pedestrians

B is the width of the deck

L is the length of the deck

The comfort classes are four with comfort class 1 as the class with highest demands on the comfort. The requirements are presented as acceleration limits and the comfort classes with corresponding limits are given in Table 5.

Table 5 Suggested comfort classes for dynamic design of lightweight footbridges (Heinemeyer et al., 2009).

Comfort class	Degree of comfort	Vertical limit [m/s^2]	Lateral limit [m/s^2]
CL 1	Maximum	< 0,50	< 0,10
CL 2	Medium	0,50 - 1,00	0,10 – 0,30
CL 3	Minimum	1,00 – 2,50	0,30 – 0,80
CL 4	Unacceptable	> 2,50	> 0,80

3.3 Regulations according to ISO 10137

The International Organization for Standardization (ISO) develops several international standards in many different technical fields. In the field of structural engineering there are a number of standards that are related to the Eurocodes. In the design of vibration serviceability there is one standard of particular interest, namely ISO 10137 Basis for design of structures – Serviceability of buildings and walkways against vibrations (Anon., 2012).

Also in the ISO 10137 standard the vibrations of footbridges are based on different design situations occurring during the design life of the bridge. The specified design situations from the standard are specified in Table 6. The second design situation intends to symbolise the regular use of the bridge while the fourth situation might never be relevant in the design (Anon., 2007).

Table 6 Design situations to consider when estimating the vibrations of a footbridge (Anon., 2007).

Design situation	Pedestrian traffic
1	Single pedestrian walking across the bridge while another pedestrian is standing in the mid-point
2	A group of eight to fifteen pedestrians crossing the bridge. The amount of people depends on the size of the bridge deck
3	Streams of pedestrians, much more than fifteen pedestrians
4	Exceptionally heavy traffic

The comfort criteria given in ISO 10137 state a maximum limit for the acceleration in vertical and lateral direction respectively. The limits for each direction depend on a specific base curve multiplied with a certain factor. The base curves for vertical and lateral accelerations are shown in Figure 6 (Anon., 2007).

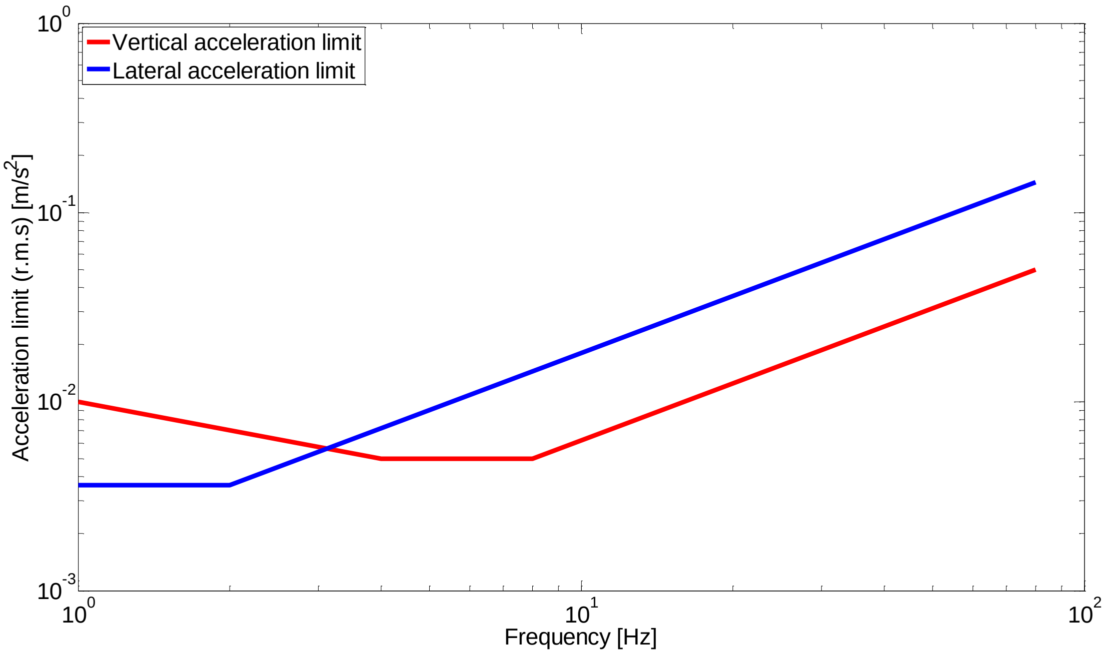


Figure 6 Base curves for lateral and vertical vibration limits according to ISO 10137. The accelerations are presented as root mean square values (Anon., 2007).

The value for the vertical vibrations of a footbridge should not exceed the values from the vertical base curve multiplied with a factor of 60. One exception is the first design situation, where one person is standing still on the bridge and a factor of 30 should be used instead. The limit values for lateral accelerations are given by multiplying the lateral base curve with a factor of 60. The corresponding limit curves for vertical and lateral vibrations are shown in Figure 7 (Anon., 2007).

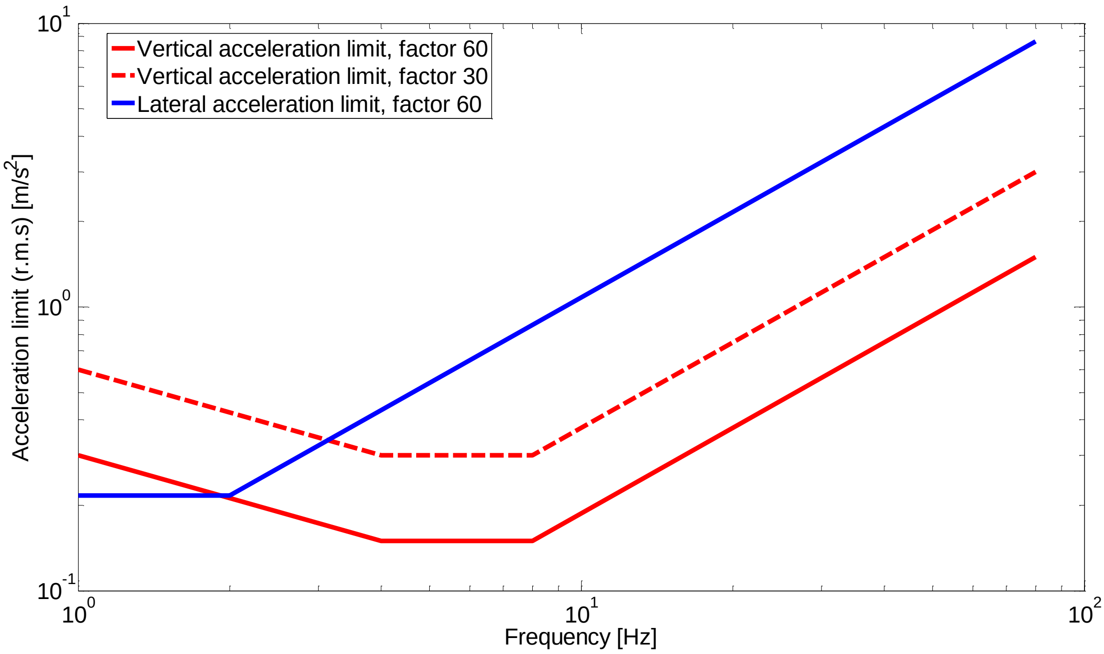


Figure 7 Acceleration limits for vertical and lateral accelerations with corresponding multiplication factors for footbridges according to ISO 10137. The accelerations are presented as root mean square values (Anon., 2007).

3.4 Regulations according to the Danish standard Belastnings- og beregningsregler for vej- og stibroer

In the Danish standard Belastnings- og beregningsregler for vej- og stibroer the limit of the peak-acceleration for vertical vibration is given in Equation (3).

$$a_{peak} < 0.25 f_0^{0.78} \tag{3}$$

where

a_{peak} is the peak acceleration limit [m/s^2]

f_0 is the frequency of the bridge deck [Hz]

If the maximum acceleration of the bridge deck is below this value the pedestrian comfort is ensured. In another clause it is stated that the comfort criteria regarding bridge sway are automatically fulfilled if the first natural frequency of the bridge deck is above 5 Hz. Regarding a maximal limit for lateral accelerations the same method as descried for vertical accelerations should be used. The annex also states critical natural frequency intervals for footbridges where resonance could occur, see Table 7 (Anon., 2002a).

Table 7 Critical intervals for natural frequencies in vertical and lateral direction (Anon., 2002a).

Frequency [Hz]	Origin of force	Direction
1.6-2.4	Walking	Vertical
2.5-4.5	Running or higher harmonics from walking	Vertical
0.8-1.2	Walking, half the vertical walking frequency	Lateral
2.6-3.4	Running	Lateral

3.5 Regulations according to the British Standard BS 5400

The British standard BS 5400 comprise a number of standards regarding the design of steel, concrete and composite bridges which was the previous standard regarding bridge design and construction in the UK, now replaced with the Eurocodes (Anon., 2010). In conjunction with the BS 5400 a standard called BD 29/04 Design criteria for footbridges was used in dynamic design (Anon., 2004a).

If the natural frequency for the bridge deck in vertical direction is above 5 Hz the comfort criteria are automatically fulfilled and no further investigations are required. The same apply if the first natural frequency in lateral direction, but for a limit of 1.5 Hz. However, if the vertical natural frequency is equal to or below 5 Hz the comfort criteria set in Equation (4) should be fulfilled (Anon., 2004a).

$$a_v < 0.5\sqrt{f_0} \tag{4}$$

where

a_v is the acceleration limit in vertical direction [m/s^2]

f_0 is the natural frequency in vertical direction [Hz]

Limits for vibrations in lateral direction are not specified in the code. Instead, it is stated that suitable limits should be set together with appropriate authority. Noted is also that the designer should be aware of that bridges with low mass and stiffness in combination with a natural frequency below 1.5 Hz could experience large lateral vibrations which should be prevented (Anon., 2001).

4 Pedestrian force models

In Chapter 2, the dynamic forces induced by pedestrians and its influencing factors are described. The irregularity of the force, e.g. that the force vary both in time and space, the synchronization phenomena and the different perception levels for each individual pedestrian are some of the factors making the force complicated to model mathematically. Though, in literature the force created by pedestrians is often modelled as a perfectly periodic force depending on the walking frequency and time (Živanović et al., 2005).

This chapter describes the force models for pedestrian loading presented in BRO 2004, Eurocode 5 and ISO 10137. In addition, some force models from research literature are also described for comparison.

4.1 Force model according to BRO 2004

The force model from BRO 2004 for pedestrian loading on footbridges is presented in Equation (5). The load is a harmonic vertical concentrated force which should be placed in the most critical point of the bridge deck, e.g. the position that results in the highest vertical acceleration. When designing according to BRO 2004 a damping factor of 0.6% of critical damping should be used (Anon., 2004d).

$$F = k_1 k_2 \, sin(2\pi f_F t) \tag{5}$$

$$k_1 = \sqrt{0.1BL} \tag{6}$$

$$k_2 = \begin{cases} 150 \; if \; f_F \leq 2.5 \; Hz \\ \frac{125}{f_F} \; if \; 2.5 \; Hz < f_F < 3.5 \; Hz \end{cases} \tag{7}$$

where

F	is the pedestrian force [N]
k_1	is a bridge size coefficient [m]
k_2	is a frequency coefficient [N]
f_F	is the natural frequency of the bridge [Hz]
t	is the time [s]
B	is the width of the bridge [m]
L	is the length of the bridge [m]

The two coefficients k_1 and k_2 presented in Equation (6) and Equation (7) describe some characteristics of the pedestrian force. The size of the bridge and hence the possible amount of pedestrians on the bridge are considered by the variable k_1. The coefficient k_2 takes the variation of the dynamic force amplitude into account. It is stated that the largest observed effect is for frequencies below 2.5 Hz. For the frequencies above 2.5 Hz the coefficient includes effects from higher harmonics. No model for the lateral force is given in the code (Anon., 2004d).

4.2 Force model according to Eurocode

In Eurocode 5, a simplified method for calculating vertical vibrations in timber bridges is given. The method applies for simply supported or truss bridges, but not to other bridge types. Instead, it is stated that other methods may be specified in the national annex or for each individual project. In Eurocode 5 the damping factor for structures with mechanical joints are set to 1.5%, which can be used if no other value is stated in the national annex (Anon., 2004c).

4.2.1 National annexes to Eurocode

In the Swedish national annex (Anon., 2011), no force model for the pedestrian force is given. Instead, it is the client that should choose the appropriate force model for the dynamic force, in the same way as for the comfort criteria (Anon., 2011).

4.3 Force model according to ISO 10137

In the international standard ISO 10137 the force from a pedestrian is described with the static weight of the pedestrian and the corresponding periodic dynamic contribution, see Equation (8). Both the vertical and lateral component of the dynamic pedestrian force is represented with this equation (Anon., 2007).

$$F(t) = Q\left(1 + \sum_{n=1}^{k} \alpha_n \, sin(2\pi n f t + \phi_n)\right), \qquad n = 1,2, \dots, k \tag{8}$$

where

$F(t)$	is the pedestrian force, lateral or vertical [N]
Q	is the static load from pedestrian [N]
n	is the integer number of the natural harmonic
k	is the total number of harmonics of interest
α_n	is a numerical coefficient
f	is the frequency of the loading [Hz]
t	is the time [s]
ϕ_n	is the phase angle for the n:th harmonic

The difference between vertical and lateral direction is the numerical coefficient α_n. In Table 8 the values for the two coefficients for one moving pedestrian are presented. Also noted in the standard is that the lateral numerical coefficient do not take the lock-in phenomena into account (Anon., 2007).

Table 8 The vertical numerical coefficient $\alpha_{n,v}$ and the lateral numerical coefficient $\alpha_{n,h}$ for one person (Anon., 2007).

	Harmonic number, n	Numerical coefficient, vertical direction, $\alpha_{n,v}$	Numerical coefficient, lateral direction, $\alpha_{n,h}$
Walking	1	0.37(f -1.0)	0.1
	2	0.1	-
	3	0.06	-
	4	0.06	-
	5	0.06	-
Running	1	1.4	0.2
	2	0.4	-
	3	0.1	-

The action from a group consisting of N people crossing the bridge can be expressed by multiplying a coordination factor to the total pedestrian force. The coordination factor aims to describe the amount of people in the group that tend to walk with the same pace. The coordination factor is expressed in Equation (9) and in Equation (10) the total force from N people is shown (Anon., 2007).

$$C(N) = \frac{\sqrt{N}}{N} \tag{9}$$

$$F(t)_N = F(t) \cdot C(N) \tag{10}$$

where

N	is the number of people in the group
$C(N)$	is the coordination factor
$F(t)$	is the total pedestrian force [N]
$F(t)_N$	is the total force with respect to coordination [N]

4.4 Force model according to the Danish standard Belastnings- og beregningsregler for vej- og stibroer

In the Danish standard Belastnings- og beregningsregler for vej- og stibroer a vertical force model with a pulsating point load and a constant forward speed is presented according to Equation (11) and Equation (12) respectively. The amplitude value of 360 N represents two small pedestrians walking together at the bridge. For substantially larger groups of pedestrians, a higher value of the amplitude should be used, especially for natural frequencies between 1.3 and 2.7 Hz. For an appropriate value, the designer is referred to the literature. The Danish standard proposes a damping factor for timber constructions that is 1.2% (Anon., 2002a).

$$F = 360\, sin(2\pi f_0 t) \tag{11}$$

$$v = 0.9 f_0 \tag{12}$$

where

F	is the pedestrian force [N]
f_0	is the natural frequency of the bridge [Hz]
t	is the time [s]
v	is the velocity of the pedestrian [m/s]

If the natural frequency is larger than 4 Hz, the calculated maximum acceleration can be reduced by a factor that varies linearly from 0% reduction at 4 Hz to 70% reduction at 5 Hz. No force model for the lateral force is presented in the annex (Anon., 2002a).

4.5 Force model according to British Standard BS 5400

In the British code BS 5400 a general method for modelling the vertical dynamic force is given that resembles the one described in the Danish standard. The force model is described in Equation (13). The pedestrian force should be applied at the bridge deck with the velocity described in Equation (14) (Anon., 2001).

$$F = 180 \sin(2\pi f_0 t) \tag{13}$$

$$v = 0.9 f_0 \tag{14}$$

where

F	is the pedestrian force [N]
f_0	is the natural frequency of the bridge [Hz]
t	is the time [s]
v	is the velocity of the pedestrian [m/s]

The only difference between the Danish model and this is the amplitude. In this case the amplitude symbolises one person walking on the bridge instead of two (Anon., 2001).

4.6 Force models presented in research literature

In research literature a number of other force models than those presented above are described. These models attempt to take different characteristics of force into account, such as the synchronization phenomena and the irregularity of the force.

According to Occhiuzzi et al. (2008) the model for the vertical pedestrian force presented in ISO 10137 characterises the walking force reasonably good, but for running it fails to describe the irregularity of the force. During walking one foot is always in contact with the ground, but for running there is a time interval every period where the person does not have contact with the ground meaning that the force becomes discontinuous. In Equation (15) and Equation (16) a force model is presented where each step is divide in a period of contact with the ground and a period without contact (Occhiuzzi et al., 2008).

$$F(t) = A \cdot m \cdot g \cdot sin\left(\frac{\pi f}{k_0}\, t\right), \qquad i \cdot T \; < \; t \; \leq (i + k_0) \cdot T \tag{15}$$

$$F(t) = 0, \qquad (i + k_0) \cdot T < t \; \leq (i + 1) \cdot T \tag{16}$$

where

$F(t)$	is the pedestrian force [N]
A	is a load amplification factor
m	is the mass of the runner [kg]
g	is the gravity constant [m/s^2]
f	is the stepping frequency [Hz]
t	is the time [s]
i	is the number of steps
T	is the period of a step
k_0	is a factor expressing the contact time

The human nature of synchronizing with the lateral movement of the bridge deck is another aspect of the dynamic pedestrian force that is taken into account in the two force models presented below. The force model in Equation (17) by Nakamura and Kawasaki (2009) takes this aspect into consideration by the synchronization function presented in Equation (18).

The function $H(\dot{u}_B)$ takes into account how one pedestrian synchronise with the girder movement depending on the velocity of the girder. If the velocity of the girder becomes too high, the pedestrians adapt their walking frequencies or grabbing the hand rail to decrease the girder velocity and hence stop the synchronization with the bridge. This is considered by the synchronization function since the function has a linear behaviour for small velocities resulting in a certain increase rate of the synchronization. However, for higher velocities the increase rate of the synchronization function decreases.

$$F = k_4 k_5 H(\dot{u}_B) G(f_0) M_P g \tag{17}$$

$$H(\dot{u}_B) = \frac{\dot{u}_B(t)}{k_3 + |\dot{u}_B(t)|} \tag{18}$$

where

F	is the pedestrian force [N]
$\dot{u}_B$	is the velocity of the bridge [m/s]
$H(\dot{u}_B)$	is a pedestrian girder movement synchronization function
f_0	is the natural frequency of the bridge [Hz]
$G(f_B)$	is a pedestrians frequency synchronization coefficient
$M_P g$	is the modal self-weight of the pedestrian
k_3	is a numerical coefficient

k_4	is the lateral force / pedestrian weight ratio
k_5	is a girder vibration synchronization coefficient
t	is the time [s]

The Millennium Bridge in London is one of the most famous bridges which have had problems with lateral vibrations due to crowd synchronization. Dallard et al. (2001b) have presented a force model for the lateral dynamic pedestrian force which is proportional to the lateral movement of the bridge. In Equation (19) the total lateral force from h pedestrians is shown.

$$F = \sum_{j=1}^{h} \phi_j^2 k_6 \dot{u}\,, \qquad j = 1,2,\dots,h \qquad (19)$$

where

F	is the pedestrian force [N]
j	is the pedestrian number
h	is the total number of pedestrians
ϕ_j	is the mode shape
k_6	is a bridge dependent numerical coefficient
$\dot{u}$	is the lateral velocity of the bridge [m/s]

5 Dynamic testing of timber bridges

Assessments of the dynamic behaviour of slender lightweight footbridges are necessary in the design. A dynamic model based on the finite element method is most often established for analysing and evaluating the characteristic parameters of the structure during the design phase. The dynamic parameters of interest are the natural frequency, the damping factor, the mode mass and the modal shape which together describes the dynamic performance of a structure. The main difficulty is to establish the damping factor for a composite structure. In design, an assumed value is used for the damping, but the real value cannot be calculated (Ohlsson, 1995).

The dynamic properties of a bridge can be determined by measuring the acceleration with accelerometers in a test with a known applied force e.g. by performing experimental modal testing. This chapter aims to shortly describe this method and different types of test methods that can be used for assessing the dynamic properties of a bridge.

5.1 Experimental modal analysis

The experimental modal analysis is used for determining the modal properties, such as natural frequencies, damping factors and mode shapes for a built structure. The method comprises vibration tests from which the data is analysed and frequency-response functions are established from which, in turn, the modal properties are estimated. Both the displacement and velocity of a node due to an induced force can be measured in a vibration test, but according to Craig Jr. and Kurdila (2006) the acceleration is the most common output to measure in a vibration test. To measure bridge accelerations piezoelectric accelerometers are the most commonly used device. In the test, the piezoelectric accelerometers are attached to the structure which is excited by a force. The measured accelerations are then transformed in to electric signals which are proportional to the accelerations (Craig Jr & Kurdila, 2006).

According to Ohlsson (1995), the choice of both locations of the measuring devices and the point for applying the force are important. Initial knowledge regarding the expected mode shapes and natural frequencies of the bridge are required to place the measuring devices in the positions and directions that will give the information of interest, for example nodes or the point where the displacements are the largest. If the load is applied in only one point, Ohlsson (1995) emphasizes that it is essential that the point do not coincide with a node. In Figure 8 a schematic mode shape is shown where the mid-point is a node, a point that does not move during oscillation. If the load should be placed in a node of a mode shape of a structure no excitation of this mode occurs.

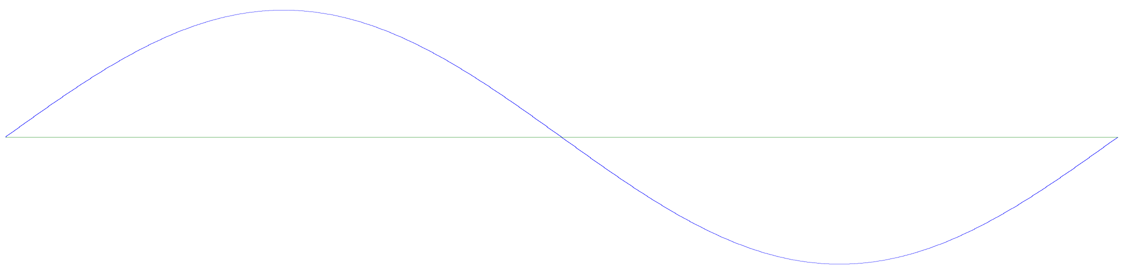


Figure 8 Schematic picture of a mode shape with a node in the middle.

Both the input and output, for example force and acceleration, in the experimental modal analysis need to be transformed from a time domain into a frequency domain. This is called Fourier transformation. When measuring a signal over a period T, it is of importance that the sampling frequency is high enough to avoid aliasing meaning that enough number of samples is measured so the signal is characterised in the right way. Aliasing means that the frequency of the signal is characterised incorrect due to peak accelerations. The Nyquist sampling theorem states that the sampling frequency should be greater than twice the maximum frequency of the sampled signal. By using the Fourier transforms of the measured input and output it is now possible to establish the frequency response functions of the bridge (Craig Jr & Kurdila, 2006).

5.2 Dynamic test methods for bridges

To create measurable accelerations in an existing bridge, a force with certain magnitude and frequency need to be applied at a certain position. All these factors depend on the dynamic properties of the bridge, which in most design are calculated with finite element software. By looking at the calculated mode shapes in advance, the point for applying the load can be decided and from the calculated natural frequencies the frequency of the applied force can be estimated (Živanović et al., 2006).

The response from the excitation is most often measured with accelerometers attached to the bridge. Below are some methods for simulating excitation loads at a bridge.

5.2.1 Impact tests

In this test, a weight is dropped at a certain point to cause an impact on the bridge. From the response, the dynamic properties of the bridge are estimated. It is of importance that no one is walking on the bridge during the test. The main advantage is that the test is repeatable, both the magnitude and the application point of the force can be exactly the same in several tests. The drawback is how the arrangement from which the load should be falling should be built up which often can be unwieldy (Jürisoo et al., 1980).

The ideal test set up is to do a snap-back test where a weight is hanged underneath the bridge in a rope. The rope is cut off and the resulting accelerations are measured. This procedure is easy to model in finite element software and therefore it is easy to verify the model. In many cases the test is not possible, instead the weight is dropped on the bridge and left on the bridge during the measurement. When analysing the results the additional mass and the changed initial conditions must be included in the calculations. Furthermore, it can be difficult to excite the bridge at the lowest frequencies with this test method (Jürisoo et al., 1980).

An easy way to do an impact test is to use the heel impact method. One person or a group of people are standing on their toes and then falling back on the heels causing impact on the bridge. The test is favourable since no special equipment is needed to cause the impact and the impact force is within a suitable frequency range that is possible to measure (Jürisoo et al., 1980).

5.2.2 Controlled walking

People walking randomly and falling in to the same pace is the normal loading situation for a footbridge and may cause vibrations in both vertical and lateral direction. In this test, a number of people are walking on the bridge to simulate a real loading situation. They can either be keeping step in walking or just walk randomly depending on how the test is designed. This test is easy to perform since no additional loading devices and test set ups are needed. The disadvantage with the test is that it is not repeatable, meaning that the exact loading frequency cannot be reproduced. It is hard to know the exact location and magnitude of the forces from all the individuals' feet and also to recreate the same walking pattern (Jürisoo et al., 1980).

5.2.3 Electrodynamic shaker

According to Živanović et al. (2005), the best method for testing is to use a shaker that is hydraulic or electrodynamic and produces a controlled force with known application point. Since it is a controlled force, the test needs shorter time for completion, which will cause fewer disturbances on the bridge traffic than other tests. Downsides with this test method are that frequencies below 1 Hz are hard to excite with enough amount of force, and the shakers are expensive, which causes the test method to become the most expensive way to test the bridge (Živanović et al., 2005).

6 Case study of Älvsbackabron

In recent years, there has been a trend towards building footbridges in cities with the motivation of decreasing motor traffic and consequently improving the air quality and living standards. Furthermore, a new bridge can be seen as a landmark and illustrate the city as modern and new thinking. This often require more creativeness and innovation by the architect and structural engineer, since the design of such a bridge can be both aesthetically demanding and technically advanced (Occhiuzzi et al., 2008).

This is the case for Älvsbackabron which is a slender cable-stayed footbridge in the centre of Skellefteå, which can be seen in Figure 9. This chapter aims to describe the structural system of Älvsbackabron and introduce the reader to the Swedish timber processing industry and describe Västerbotten as a timber promoting county. Also included is a description of the research project Smart Wooden Bridge in Smart City and its participants.

Figure 9 The slender profile of Älvsbackabron. The picture is taken from Anderstorp located on the south side of the river.

6.1 The structural system of Älvsbackabron

In a cable-stayed bridge, the bridge deck is supported by cables connected to pylons. Cable-stayed bridges are suited for design situations where large spans must be bridged without introducing new support placements. Crossing highways, railways and watercourses are examples of situations where an increasing amount of support is unfavourable (Pousette, 2001).

Älvsbackabron is a symmetrical cable-stayed bridge with one span and two pylons, shown in Figure 10. The bridge spans 130 meters over Skellefteå River creating a crossing between Anderstorp and Älvsbacka, two districts of Skellefteå. The bridge deck is slightly curved where the mid-point is about one meter above the end points.

Figure 10 Overview of the symmetric Älvsbackabron with a span of 130 meters.

The height of the pylons are 23 meters and one pylon consists of two glulam columns with four glulam cross beams and steel cross bracing to stabilise the pylon. The south pylon is shown in Figure 11 where the anchorage cables also can be seen. The cables are anchored in concrete foundations on the landside. At the top of each pylon column a total amount of six cables are connected, two anchorage cables and four cables in opposite direction supporting the bridge deck.

Figure 11 The south pylon of Älvsbackabron with its anchorage cables and concrete foundations

Figure 12 shows the bridge from below. The main beam consists of three glulam beams glued together creating one beam with the dimensions 645x1125 mm. The main beams are supported by steel cross beams every 16.25 meters to which the cables are connected at each side. Glulam cross beams are connecting the two main beams with distance of 2.5 meters. On top of the glulam cross beams purlins are placed to carry the deck of wooden planks. To stabilise the bridge deck steel cross bracing are used which also can be seen in Figure 12. The main beams of the deck and the cross beams and columns of the pylons are covered with yellow painted panels for weather protection.

Figure 12 Älvsbackabron from below. The main beams are supported by steel cross beams to which the cables are attached. The glulam cross beams, timber purlins and steel cross bracing can also be seen.

6.2 Skellefteå and the timber processing industry

The city of Skellefteå is situated in the northern part of Sweden in Västerbotten County. The timber processing industry has a traditional and important value for the county since it employs a big part of the population (Nationalencyklopedin, 2012). Martinsons AB, one of the biggest wood processing companies in Sweden, is located in the Västerbotten County and is specialised in building timber bridges and building systems for frameworks in timber for multi-storey residential houses. In addition, the company has its own saw mill and is also the largest producer of glulam products in Sweden (Martinsons, 2012).

Other important actors in the timber processing industry located in Skellefteå is the division of timber engineering of Luleå University of Technology and SP Trätek, a division of SP Technical Research Institute of Sweden focusing on wood technology. This makes the city of Skellefteå notable and important in the aspiration of increasing the use of timber and timber products as a modern and sustainable construction material (Trästad 2012, 2012).

Älvsbackabron is one of several projects with connection to the timber processing industry built in Skellefteå. Another project is Älvsbackastrand, a new residential area consisting of three multi-storey residential buildings with timber frames located at the shore of Skellefteå River adjacent to Älvsbackabron, see Figure 13. A third project is a multi-storey car park built in the centre of Skellefteå called Kvarteret Ekorren (Trästad 2012, 2012). All these projects are results from the municipality´s goal of using and increasing the competence of the timber building technology and a vision of a modern and sustainable society (Mynewsdesk, 2010).

Figure 13 Älvsbackastrand, residential buildings with timber framework close to Älvsbackabron. One of many timber projects in Skellefteå.

6.2.1 Smart Wooden Bridge in Smart City

Smart Wooden Bridge in Smart City, with the original title “Smart Träbro i Smart Stad”, is a research project carried out by the municipality of Skellefteå in association with Luleå University of Technology. Together they have appointed a project group with representatives from the commercial and industrial life, research and the public sector. SP Trätek is involved in the project as specialist and researcher and will together with Luleå University of Technology carry out the research (Degerfeldt, 2009).

The purpose of the project Smart Wooden Bridge in Smart City is to increase the competiveness in the timber processing industry and mark the region of Västerbotten as innovative and leading in the industry (Degerfeldt, 2009). One way to fulfil this purpose is to develop measuring systems for assessing the behaviour of the bridge. Gustafsson and Saracoglu[2] describe that during the erection of the bridge measuring equipment were installed by SP Trätek and Luleå University of Technology with the

[2] Anders Gustafsson and Erhan Saracoglu SP Trätek, presentation March 27:th 2012

purpose to create a database containing different measured data. Accelerometers, weather sensors, GPS systems, hygrotracs, strain gauges and a camera were installed at the bridge. From the database the performance of the bridge during pedestrian traffic, heavy weather and seasonal changes can be analysed. Furthermore, knowledge concerning methods for evaluating the quality of the structural components in a timber bridge will be gained and the understanding of how maintenance of a timber bridge should be performed will be increased (Degerfeldt, 2009).

The effects of the project do not only include aspects involving improvement and development of the timber processing industry and research in the field, but also positive effects such as decreasing the motor traffic in the centre of Skellefteå and improving the health and standard of living for the people of Skellefteå are expected results from the new bridge (Degerfeldt, 2009). A scenic picture of the bridge is shown in Figure 14.

Figure 14 The scenic view of Älvsbackabron. The new bridge is supposed to improve the living standard for the people of Skellefteå.

Martinsons Träbroar AB is a representative from the project group who will contribute to the research with time in the project group. According to Jacobsson (2009) their main interest in the research project is to increase the knowledge of the dynamical behaviour in this type of large timber bridges. By measuring the oscillations of Älvsbackabron caused by pedestrians and wind, the dynamical behaviour of the bridge can be studied and compared with the theoretical calculations. With the gained knowledge the timber bridge design can be improved and in turn strengthen timber bridges as a competitive bridge type. There are important environmental advantages in increasing the understanding and competence in timber bridge design. One benefit in using timber as a construction material is that the material is a local and renewable resource. An advantage of the bridge type is that it has little influence on watercourses, both during construction and when finished (Jacobsson, 2009).

Smart Wooden Bridge in Smart City is partly financed by a contribution from the European Regional Development Fund. The contribution is within intervention 1.2 Innovation and renewability with the incentive of creating sustainable development by reinforcing the competitiveness and innovation in the region of Övre Norrland (Puranen, 2011). Five million Swedish kronor were given to the municipality of Skellefteå and Luleå University of Technology where three millions were dedicated to the research project and two millions were given to the execution of the bridge. Remaining costs for the bridge, 25 million, were paid by the municipality of Skellefteå (Byggindustrin, 2010).

7 Modelling of Älvsbackabron

In the Master's thesis, two finite element models of Älvsbackabron are established in Brigade/Plus. One of the models is established with material properties according to BRO 2004 and compared with the dynamic design model by COWI AB. The dynamic design model was made in 2010 by Thomas Hallgren[3], structural engineer at COWI AB. This model is later referred to as the dynamic design model by COWI.

The two Master's thesis models are modelled in the same way, but with material parameters which are based on either BRO 2004 or Eurocode 5. These models are referred to as Master's thesis model according to BRO 2004 and Master's thesis model according to Eurocode. In this chapter the dynamic design by COWI will be presented followed by the Master's thesis models.

7.1 Structural model of the bridge

In both the dynamic design model of the bridge and in the Master's thesis models the same kind of elements is used. The main beams, pylons and the cross beams in pylons and the deck are modelled using beam elements. The deck is modelled as shell elements, where the material stiffness and weight of the purlins and timber planks are weighted to simulate the real arrangement and behaviour. The third element used in the model is truss elements for the cables and cross bracing. The structural system of the bridge is illustrated in Figure 15.

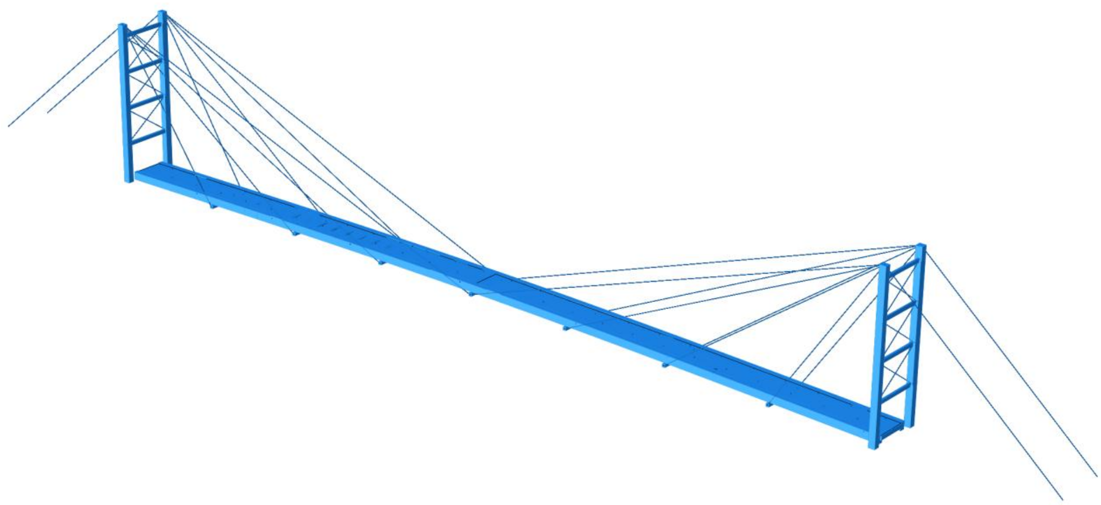


Figure 15 Structural model of Älvsbackabron.

7.1.1 Dynamic design by COWI

The dynamic design of Älvsbackabron was carried out by COWI in Gothenburg in cooperation with the division for aerodynamics from COWI Denmark. The finite element software called NEiNastran with the pre- and postprocessor Femap was used for the dynamic analyses of Älvsbackabron. Included in these analyses are both

[3] Thomas Hallgren, structural engineer COWI AB

impact from wind and corresponding aerodynamic instability and the dynamic effects from pedestrian traffic[4]. The pedestrian induced vibrations are of main interest in this thesis, why no further considerations regarding the aerodynamic design are made.

In Table 9, the input data from the dynamic design model by COWI are shown. The same data are later used for verifying the Master's thesis model according to BRO 2004.

Table 9 Input data from the dynamic design model by COWI and the Master's thesis model according to BRO 2004.

	Young's modulus [GPa]	Poisson's Ratio	Density [kg/m^3]	Shear modulus [GPa]
Deck	4.846	-	454	0.323
Glulam	13	0.4	600	4.643
Steel	210	0.3	7850	80.769
Cable	200	0.3	7850	76.923

7.1.2 Simplifications and assumptions in the Master's thesis models

The Master's thesis models are created with the finite element software Brigade/Plus and all dynamic analyses are performed with linear analysis. In the models a number of simplifications of the bridge structure are done.

In reality the cables are attached to the top of the pylons with steel attachments, which can be seen in Figure 16. In the model, however, all cables are attached directly to the pylon tops. The simplification results in a small change of angle for the cables, but these effects are considered negligible.

Further simplifications related to the pylons are regarding the two anchorage cables which in one end are attached to the column top and in the other end are attached to the concrete foundation. The connection to the top of the pylon is seen in Figure 16 and to the concrete foundation in Figure 17. The cables are modelled as one cable with an increased cross section area corresponding to the total area of the two cables.

[4]Thomas Hallgren, structural engineer COWI AB, meeting March 21:th 2012

Figure 16 The pylon top to which the anchorage cables are attached to. Simplifications are made in the model both regarding the steel attachment and the anchorage cables.

The connection of the anchorage cables to the concrete foundation is not modelled either. Instead, the end of each modelled cable is restrained to move in all directions in the model.

Figure 17 Connection of the anchorage cables to the concrete foundation at the north side of the bridge. The cables are modelled as one cable with a cross section area corresponding to the total area of the two cables.

The pylons are in reality connected with steel connections to the concrete foundations which in turn are piled to the ground. Both the connection between pylon and concrete foundation and the foundation to the piles are simplified in the model. The support conditions are modelled with spring element which simulates the effect from the piles. In the dynamic design model by COWI the piles are modelled, but the difference between piles and spring elements are negligible[5]. The spring elements are connected directly to the pylons meaning that the steel connection is excluded in the model. The real connection can be seen in Figure 18.

Figure 18 The connection between one pylon and the concrete foundation. In the finite element model the steel connection is excluded.

Other assumptions in the model are the connections between the cross beams and the main beams. These connections are modelled as rigid connections, but in reality they consist of steel connectors and screws which are not fully rigid. The deck is, as described previously, modelled as a shell with weighted material parameters. For a more detailed description of the modelling process in Brigade/plus, see Appendix A.

7.2 Master's thesis models

The Master's thesis model according to BRO 2004, based on material parameters from BRO 2004, is verified with the dynamic design model by COWI by using the same material parameters. The reason for this is to avoid possible modelling mistakes in the Master's thesis models and insure that the differences in results between the two Master's thesis models do not depend on modelling mistakes, but instead differences between the codes.

Some results from the verification of the Master's thesis model according to BRO 2004 are shown in Table 10. The static deflection due to service load and the first natural frequencies of the bridge deck are compared for the dynamic design model by COWI and the Master's thesis model according to BRO 2004. The differences between the models are considered reasonably small why the Master's thesis models are considered reliable.

[5]Thomas Hallgren, structural engineer COWI AB, meeting March 21:th 2012

Table 10 Verification of Master's thesis model according to BRO 2004 with the dynamic design model by COWI by comparing some results from the models.

Comparison element	COWI	Master's thesis model according to BRO 2004
Deflection due to uniformed pedestrian load 4 kN/m^2	166 mm	163 mm
First lateral natural frequency	0.614 Hz	0.620 Hz
Second lateral natural frequency	1.415 Hz	1.386 Hz
First vertical natural frequency	1.448 Hz	1.441 Hz
Second vertical natural frequency	1.902 Hz	1.899 Hz

The only differences between the two Master´s thesis models are the material properties given in the codes. The material properties that are used in the Master's thesis model according to Eurocode are presented in Table 11.

Table 11 Input data for Master's thesis model according to Eurocode.

	Young's modulus [GPa]	Poisson's Ratio	Density [kg/m^3]	Shear modulus [GPa]
Deck	5.923	-	259	0.372
Glulam	13.7	0.4	430	4.893
Steel	210	0.3	7850	80.769
Cable	210	0.3	7850	80.769

The force models for the dynamic pedestrian force presented in BRO 2004 and ISO 10137 are simulated in both the models and the results from Brigade/Plus is presented in Section 9.1 and Section 9.2.

The force model given in ISO 10137 consists of a Fourier series which takes the contribution from higher harmonics into account by introducing a phase angle. The force model can be seen in Equation (20). A recommended value for the phase angle is a shift of 90˚ (Anon., 2007). The resulting force is periodic, but not harmonic due to the phase shifts.

$$F(t) = Q(1 + \sum_{n=1}^{3} \alpha_n \, sin(2\pi nft + \phi_n)), \qquad n = 1,2,3 \qquad (20)$$

To be able to apply the vertical force model given in ISO 10137 over a frequency interval in Brigade/Plus, the force has to be harmonic. In Figure 19, the force model in Equation (20) with phase shift and the simplified harmonic force model are shown. To simplify the force model the phase shift is neglected, but the magnitude of the first three vertical harmonics are still considered by taking their contribution by α_n into account. The lateral force model only takes the first harmonic into account and therefore no simplification is needed.

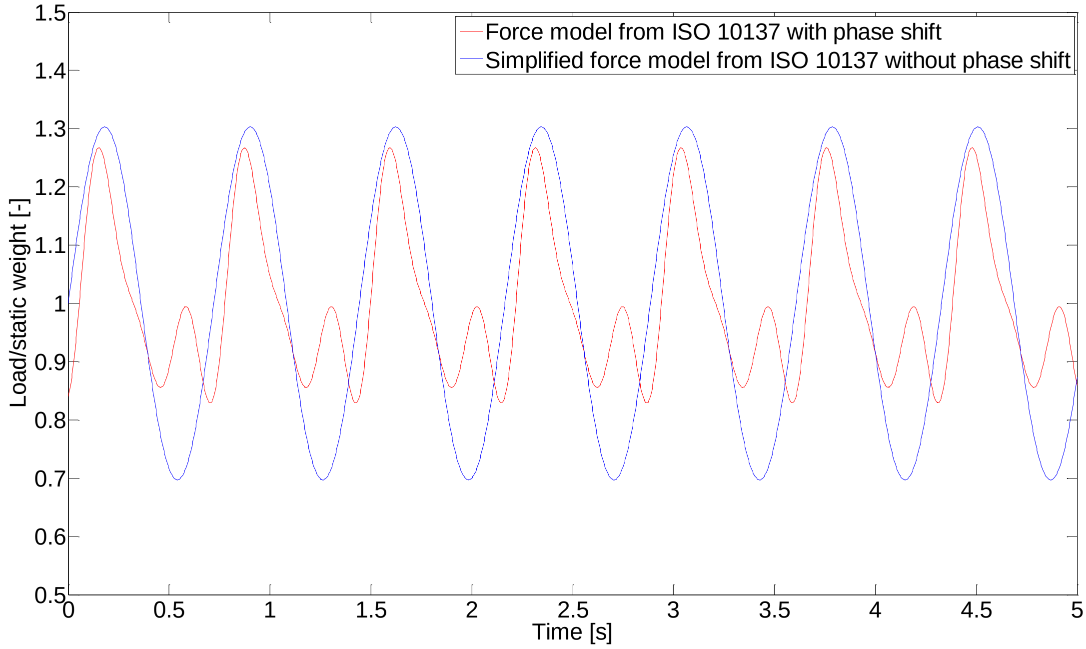


Figure 19 *Force model according to the ISO 10137 standard with a phase shift of 90° for the second and third harmonic as presented in Equation (20). Furthermore, a simplified force model without the phase shift is described. The simplification results in a harmonic force.*

The force amplitudes for the different force models are presented in Table 12. In the ISO 10137 model the static weight of a pedestrian is included in the force amplitude, which in this study is assumed to be 80 kg. The sum of the three first α_n-factors that are used in the simplified ISO 10137 force model is 0.3.

One of the recommended design situations in ISO 10137 is a stream of pedestrians significantly larger than fifteen pedestrians. In the simplified force model, a group of fifty pedestrians is chosen, which means that $\sqrt{50}$ pedestrians are assumed to walk with the same pace.

In BRO 2004 no force model regarding lateral vibrations is given. Instead, as described in Section 2.1.1, 10% of the vertical force is assumed to act as a lateral force in the calculations. The different loads are all harmonic periodic forces and the resulting accelerations will be calculated over a frequency sweep from 0 to 5 Hz.

Table 12 Force amplitudes for the dynamic pedestrian forces used in Section 9.1 and Section 9.2. These values are used for calculating accelerations in the bridge deck of the two models.

Code	Vertical force amplitude [N]	Lateral force amplitude [N]
BRO 2004	1082	108
ISO 10137, 1 runner	1520	160
ISO 10137, 1 pedestrian	240	80
ISO 10137, 10 pedestrians	760	250
ISO 10137, 50 pedestrians	1700	565

8 Acceleration measurements at Älvsbackabron

The accelerations in the bridge deck at Älvsbackabron are measured at two occasions. As an interesting detail it can be mentioned that the local newspaper Norran made an article about the second occasion and the research project Smart Bridge in Smart City (Dhyr, 2012). The article can be found in Appendix B. In this chapter the equipment used for measuring at Älvsbackabron will be presented together with the performed tests at the bridge. In the description of the tests some of the theoretical results are presented to motivate the frequencies used in the tests.

8.1 Measuring equipment

In this report, the accelerations in the bridge deck are of main interest. To be able to measure the accelerations of the bridge, a portable measuring device is used during the tests. This measuring device consists of piezoelectric accelerometers connected to a data acquisition module which in turn is connected to a computer. Two of the accelerometers and the data acquisition module are shown in Figure 20. With this device up to four accelerometers can be connected at the same time.

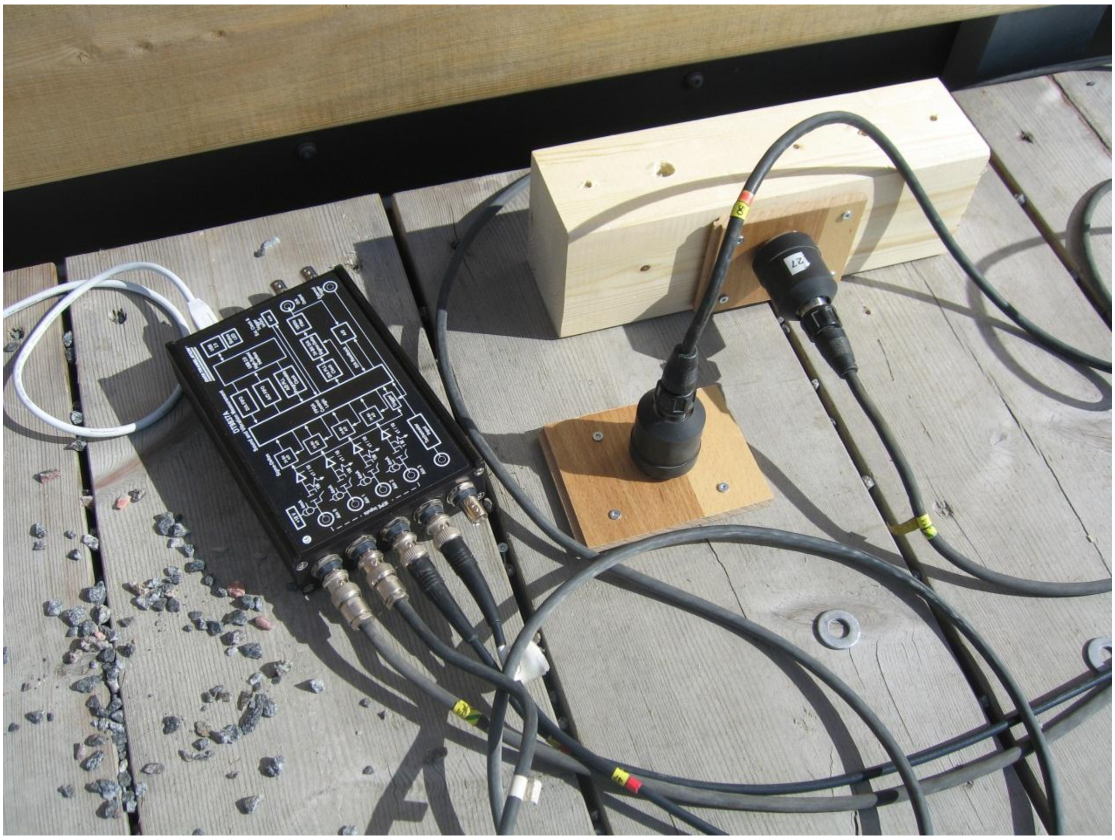


Figure 20 Portable measuring device consisting of accelerometers attached to a data acquisition module. The accelerometer mounted directly on the bridge deck measures the vertical accelerations while the other measures lateral accelerations.

During the first test occasion in March four ICP accelerometers of model number V356A11 were used and for the second test occasion in May four ICP accelerometers of model number 393B12 were used. Information and calibration certificates for the accelerometers used at Älvsbackabron are shown in Appendix C. Two of these accelerometers were mounted at the middle of the bridge span and two others at the quarter of the bridge span, according to Figure 21. At each position one accelerometer measures the accelerations in vertical direction and the other measures in the lateral direction.

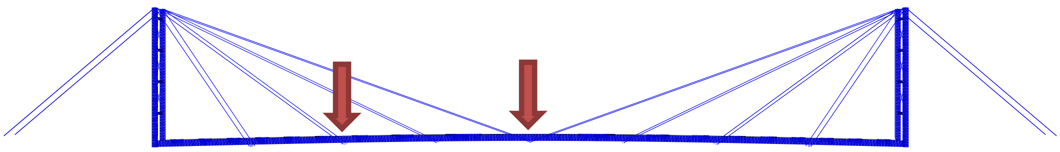


Figure 21 Middle and quarter of the bridge span where the loading and accelerometers are positioned.

8.2 Tests performed at Älvsbackabron

To verify the dynamic design of Älvsbackabron three different tests were conducted on the bridge. The tests were controlled walking, controlled jumping and heel impact. Also, continuous measurements were made to investigate the level of the accelerations during normal use of the bridge. Frequencies for the controlled walking and jumping tests were chosen according to theoretical natural frequencies calculated in the Master's thesis models of Älvsbackabron. The natural frequencies calculated with the two Master's thesis models differ slightly from each other. This is the reason why two similar frequencies are chosen to excite a specific mode in the controlled walking and jumping tests.

The tests were performed at two different occasions, one in March and one in May. During all tests, except the continuous measurements, the bridge was closed to traffic to avoid disturbances.

8.2.1 Controlled walking test

To create measureable accelerations in the bridge deck, a number of repeated controlled walking tests were performed. A description of the controlled walking tests is presented in Section 5.2.2. During the test a group of eight people walked over the bridge with the same pacing rate. The different paces were indicated by a metronome and in Table 13 the tested walking frequencies are presented and the corresponding mode shapes that were supposed to be excited. The natural frequency of the lateral mode is half the walking frequency due to the fact that only alternate step give rise to a lateral force in each direction.

Table 13 Walking frequencies for the controlled walking tests and their corresponding mode shapes.

Walking frequency [Hz]	Excited mode shape
1.4	First vertical and lateral
1.5	First vertical and lateral
1.9	Second vertical

Figure 22 shows one of the walking tests at Älvsbackabron in March. The controlled walking tests are performed with the intention of simulating possible loading scenarios that can arise on the bridge.

Figure 22 One of the walking tests at Älvsbackabron with controlled walking frequency.

8.2.2 Jumping test

The jumping test aims to simulate the performance of an electrodynamic shaker, which was described in Section 5.2.3. The group of eight people jumped in a certain frequency to create a periodic load that acted on one point of the bridge. A picture of one jumping test is shown in Figure 23.

Figure 23 One of the jumping tests performed at Älvsbackabron (Eriksson, 2012).

To excite the first vertical mode, the jumping group was positioned in the middle of the bridge and to excite the second vertical mode the group jumped in the quarter of the bridge span, see Figure 21. The jumping positions, frequencies and expected mode shapes are presented in Table 14. From the tests the acceleration can be evaluated and a damping factor for the bridge can be calculated.

For all the jumping tests the group jumps for about 30 seconds and afterwards remains stationary for about 30 seconds.

To excite the lateral mode ice skating jumps are performed at the middle of the bridge. The frequency for this type of jumps is twice the expected natural frequency since only every second jump gives rise to a lateral force in each direction. The expected natural frequency is around 0.7 Hz and therefore the jumping frequency is chosen to 1.4 Hz, see Table 14.

Table 14 Jump direction, position and frequency together with the expected mode to excite.

Jump direction	Load position	Jumping frequency [Hz]	Mode
Vertical	L/2	1.4	First vertical
Vertical	L/2	1.5	First vertical
Vertical	L/4	1.9	Second vertical
Vertical	L/4	2.1	Second vertical
Lateral	L/2	1.4	First lateral

8.2.3 Heel impact test

To induce a simple impact load on the bridge as described in Section 5.2.1 the heel impact test is performed. The heel impact test was performed at the bridge in the midspan and in the quarter of the span. The same group as in the previously described tests were standing on their toes close together and falling back on their heels at the same time. At each test, this procedure was repeated two times with 30 seconds interval during which all the test participants were standing still. The purpose of the test is to estimate a damping factor.

8.2.4 Continuous measurements

The accelerations of the bridge deck during the regular use of the bridge are of interest when evaluating the performance of the bridge. To obtain measurements reflecting the regular use of the bridge measurements at Älvsbackabron were conducted during two afternoons with the accelerometers placed in the middle and the quarter of the bridge deck. From these tests, the largest accelerations due to normal use can be measured.

8.3 Simulation of tests in Brigade/Plus

The jumping tests and the heel impact tests are simulated in the Master’s thesis models in Brigade/Plus to achieve theoretical data for the comparison. The theoretical forces from the tests are presented together with other relevant input data.

The jumping tests are modelled in Brigade/Plus with a pulsating force with the shape of a half-sine curve. A graph of the force is shown in Figure 24. The maximum amplitude of the dynamic force is assumed to be three times the static weight of the group (Bachmann & Ammann, 1987). The static weight of the participating people is 730 kg which is used in the simulations. In the example presented in Figure 24 the force frequency is 1.4 Hz, the same as the test described in Section 8.2.2 and the load is applied in midspan of the model.

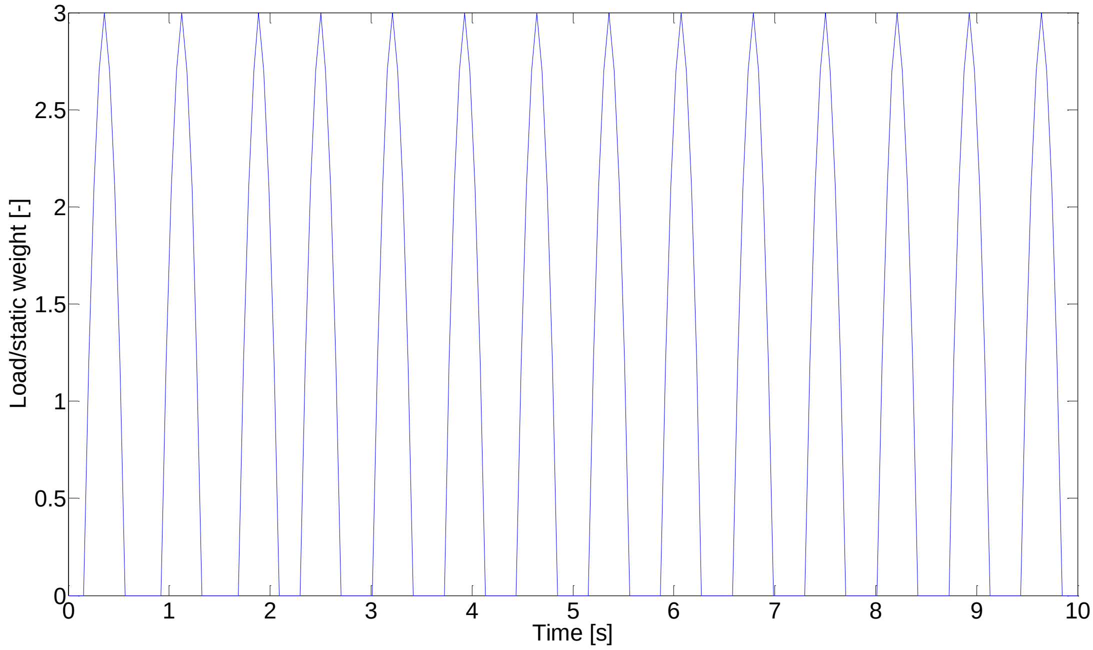


Figure 24 Half-sine force factor to simulate jumping force induced by the group.

The heel impact test is simulated with a point load applied in the midspan. The magnitude of the force is assumed to be twice the static force from the participating group, in the range between the magnitude of walking and jumping force. The force and its duration are shown in Figure 25.

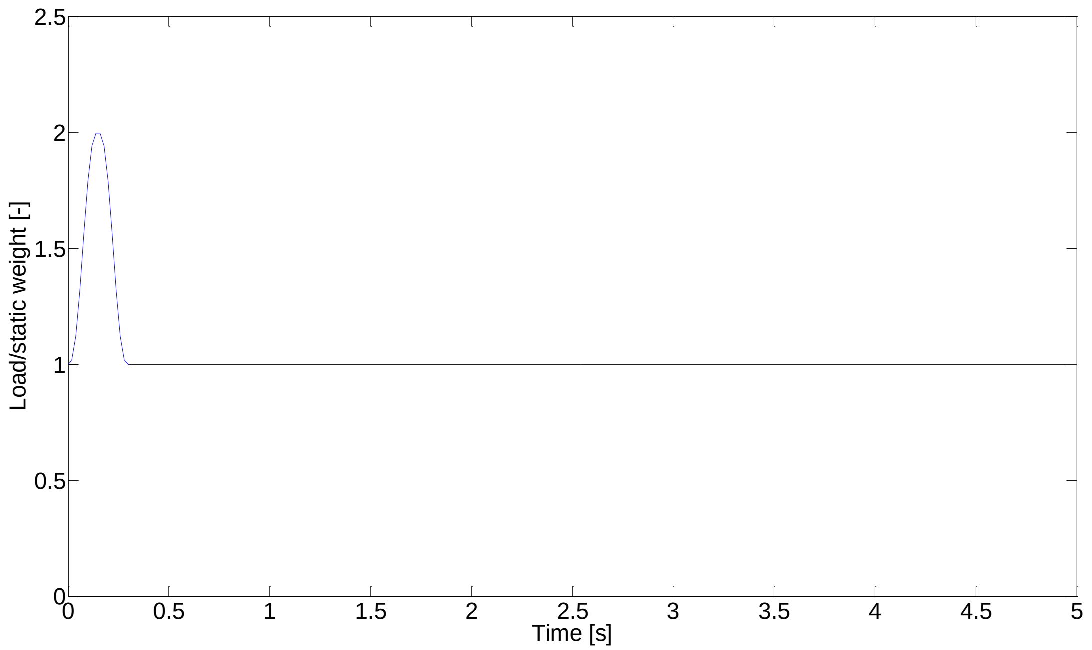


Figure 25 The heel impact force simulation in Brigade/Plus. An impact force twice the static force is applied during 0.3 s.

9 Results

In this chapter, the results from the both Master's thesis models are presented. The natural frequencies and the accelerations resulting from the force models in BRO 2004 and ISO 10137 are presented. Moreover, the acceleration measurements from the different tests at Älvsbackabron are also presented with corresponding results from the force simulations in Brigade/Plus.

9.1 Master's thesis model according to BRO 2004

The Master's thesis model according to BRO 2004 with its input data is described in Section 7.1.1. In Table 15, the first ten natural frequencies of the model are presented. The corresponding mode shapes are included in Appendix D.

Table 15 The first ten natural frequencies and description of mode shapes from the Master's thesis model according to BRO 2004.

Mode number	Description of mode shape	Natural frequency [Hz]
1	Lateral movement of bridge deck	0.620
2	Vertical movement of bridge deck	1.386
3	Lateral movement of pylon	1.390
4	Lateral movement of bridge deck	1.441
5	Lateral movement of pylon	1.479
6	Vertical movement of bridge deck	1.899
7	Torsional movement of bridge deck	2.350
8	Lateral movement of bridge deck	2.559
9	Longitudinal movement of pylons	2.760
10	Longitudinal movement of pylons	2.786

In Figure 26, the resulting vertical accelerations from the Master's thesis model according to BRO 2004 with the force models in BRO 2004 and the simplified force model from ISO 10137 are presented. The simplification of the force model in ISO 10137 is described in Section 7.2. Both the force models in BRO 2004 and in ISO 10137 are time dependent, but with a built in function in Brigade/Plus the force is converted to a frequency domain. The forces have been applied with a frequency sweeping from 0 to 5 Hz in the midspan of the bridge. The peaks with the highest accelerations indicate the natural frequencies, i.e. where resonances occur. The first vertical mode of the bridge deck gives the highest acceleration, why it is reasonable to expect the highest accelerations from this walking frequency. The simplified force model in ISO 10137 with a group of fifty pedestrians results in the highest accelerations. The maximum values of the vertical accelerations given in Figure 26 are presented in Table 16.

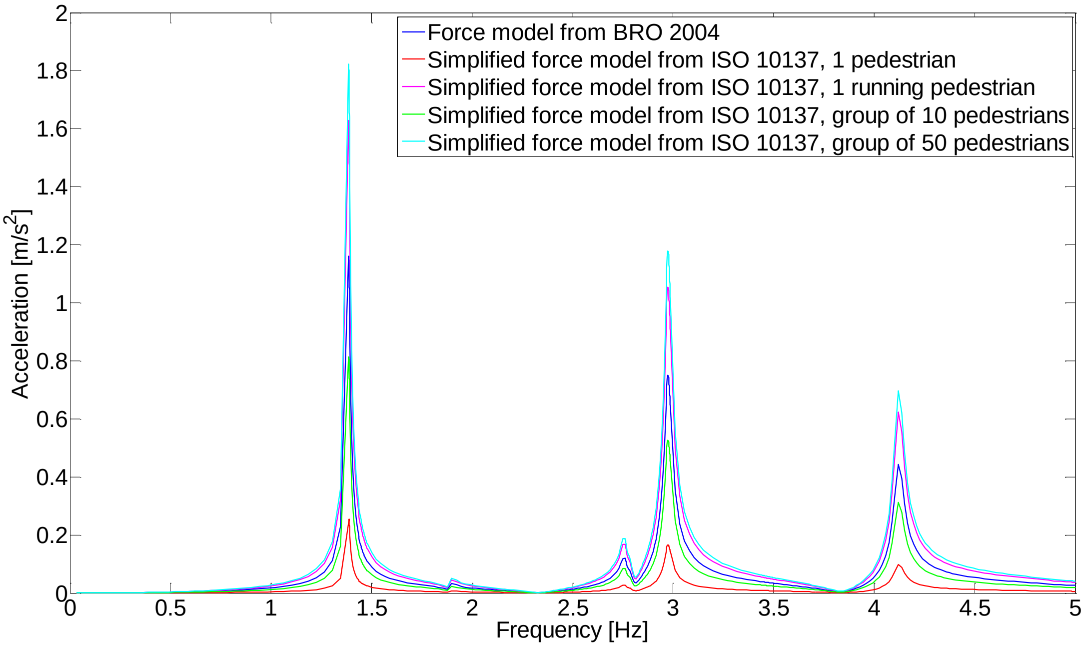


Figure 26 Vertical accelerations from force models sweeping over a frequency interval from 0 to 5 Hz. The forces are applied on the Master's thesis model according to BRO 2004. The maximum vertical accelerations for each force model are given in Table 16.

Results from force models in lateral direction are presented in Figure 27. The highest lateral accelerations appear at the third lateral mode. The maximum lateral accelerations from the applied force models are seen in Table 16. Also for the lateral accelerations it is the force model from ISO 10137 with fifty pedestrians that results in the highest values of the accelerations.

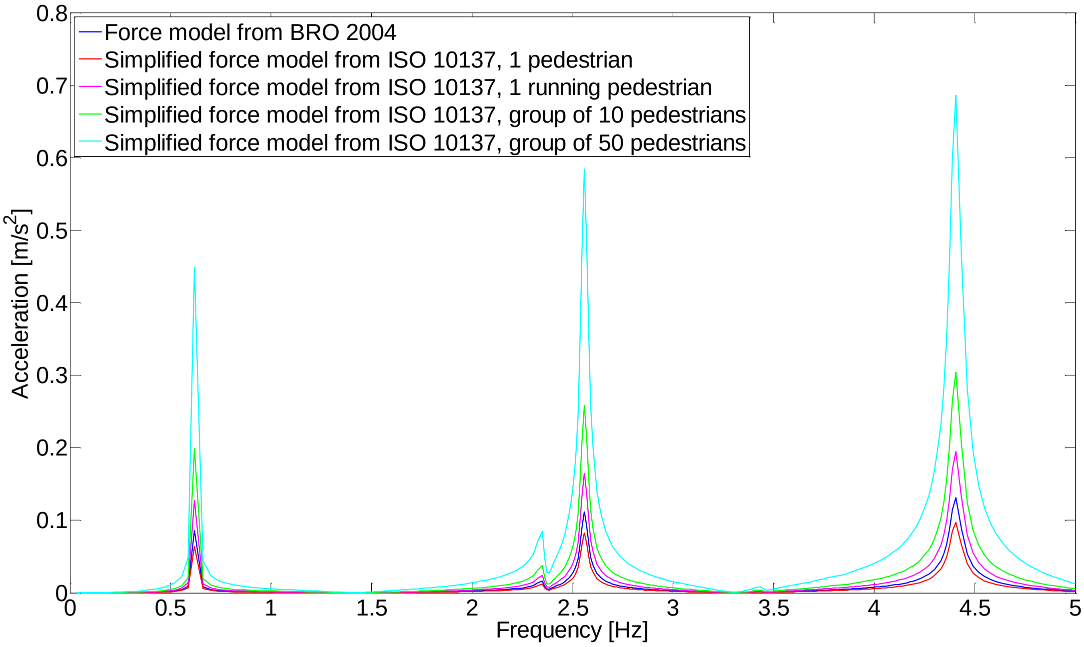


Figure 27 *Lateral accelerations from force models sweeping over a frequency interval from 0 to 5 Hz. The forces are applied on the Master's thesis model according to BRO 2004. The maximum lateral accelerations for each force model are given in Table 16.*

The ISO 10137 force model with fifty pedestrians results in the highest accelerations for both vertical and lateral accelerations. Notable is also that the maximum resulting vertical acceleration from one runner is higher than the accelerations from ten walking pedestrians and the force model given in BRO 2004.

Table 16 *Maximum vertical and lateral accelerations from force models in BRO 2004 and ISO 10137 applied on Master's thesis model according to BRO 2004.*

Force model	Maximum vertical acceleration [m/s^2]	Maximum lateral acceleration [m/s^2]
BRO 2004	1.16	0.13
ISO 10137, 1 pedestrian	0.26	0.10
ISO 10137, 1 running pedestrian	1.63	0.19
ISO 10137, 10 pedestrian	0.81	0.30
ISO 10137, 50 pedestrian	1.82	0.69

9.2 Master's thesis model according to Eurocode

Corresponding calculations as previously described are performed in the Master's thesis model according to Eurocode. An extensive description of the Master's thesis model according to Eurocode is given in Section 7.2. The first ten natural frequencies of the bridge are presented in Table 17 and the corresponding mode shapes are presented in Appendix D.

Table 17 Natural frequencies and description of the mode shapes for the first ten modes from the Master's thesis model according to Eurocode.

Mode number	Description of mode shape	Natural frequency [Hz]
1	Lateral movement of bridge deck	0.746
2	Lateral movement of pylon	1.539
3	Lateral movement of pylon	1.644
4	Vertical movement of bridge deck	1.648
5	Lateral movement of bridge deck	1.732
6	Vertical movement of bridge deck	2.285
7	Torsional movement of bridge deck	2.714
8	Lateral movement of bridge deck	3.057
9	Longitudinal movement of pylons	3.244
10	Longitudinal movement of pylons	3.274

The force models from BRO 2004 and ISO 10137 have been applied in the same way as described earlier in Section 9.1. The load is placed in the middle of the bridge with a frequency sweeping from 0 to 5 Hz. Figure 28 shows the resulting vertical accelerations and the maximum resulting accelerations can be seen in Table 18. The first natural frequency of the bridge deck gives the highest vertical accelerations.

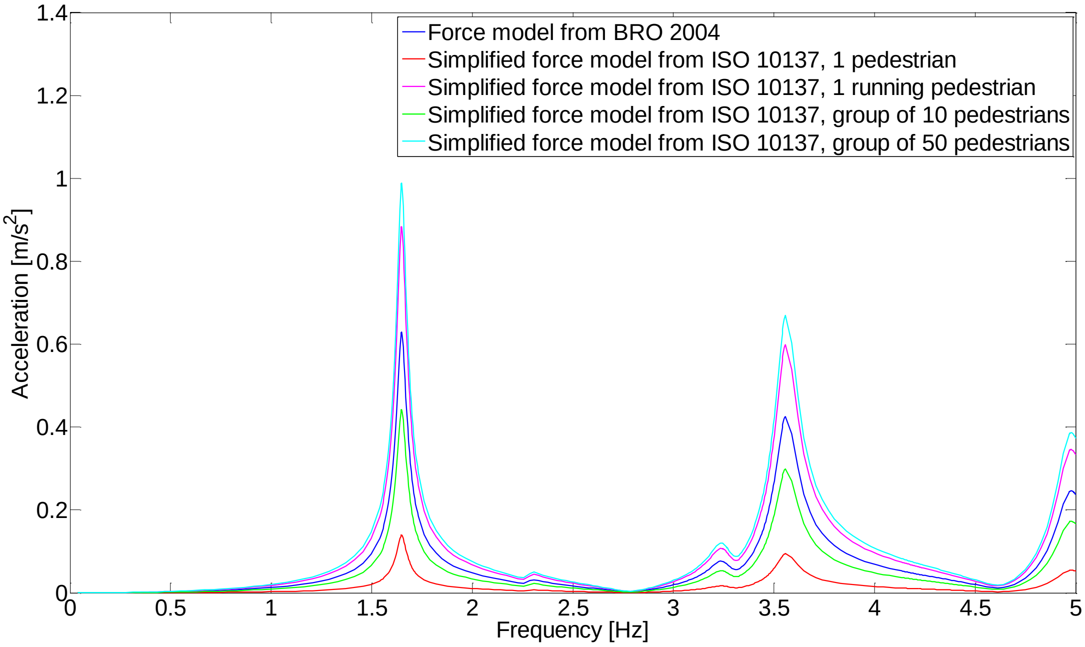


Figure 28 *Vertical accelerations from force models sweeping over a frequency interval from 0 to 5 Hz. The loads are applied on the Master's thesis model according to Eurocode. The maximum vertical accelerations for each force model are given in Table 18.*

The lateral accelerations are seen in Figure 29 and the maximum lateral accelerations are presented in Table 18. Here it is the second lateral mode that gives the highest accelerations.

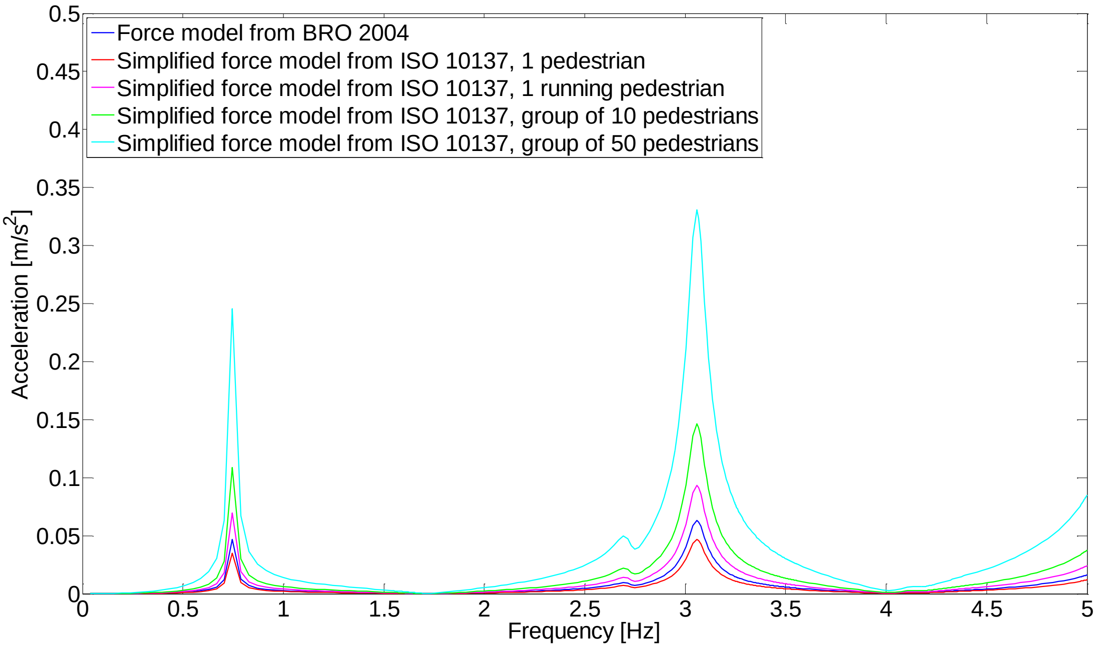


Figure 29 *Lateral accelerations from force models sweeping over a frequency interval from 0 to 5 Hz. The forces are applied on the Master's thesis model according to Eurocode. The maximum lateral accelerations for each force model are given in Table 18.*

Also in this model the resulting vertical acceleration from a runner is the second highest value, higher than a group of ten walking pedestrians. However, highest accelerations are achieved from the force model with fifty pedestrians.

Table 18 Maximum vertical and lateral accelerations from force models in BRO 2004 and ISO 10137 applied on Master's thesis model according to Eurocode.

Force model	Maximum vertical acceleration [m/s^2]	Maximum lateral acceleration [m/s^2]
BRO 2004	0.63	0.06
ISO 10137, 1 pedestrian	0.14	0.05
ISO 10137, 1 running pedestrian	0.88	0.09
ISO 10137, 10 pedestrian	0.44	0.15
ISO 10137, 50 pedestrian	0.99	0.33

9.3 Tests at Älvsbackabron

The results from the tests are presented as measured acceleration in two positions of the bridge deck, namely midspan and a quarter of the span. The measured data are filtered to only include frequencies up to 5 Hz using a built in low pass filter in MATLAB. The accelerations are plotted versus time and from these curves the resulting accelerations of the bridge are seen and the maximum accelerations are determined. From the jumping test and heel impact test damping factors are calculated according to Equation (21).

$$\zeta = \frac{1}{2\pi} \ln\left(\frac{a_i}{a_{i+1}}\right) \tag{21}$$

where

ζ	is the damping factor
a_i	is the acceleration at cycle i
a_{i+1}	is the acceleration at cycle i+1

A possible source of error influencing the measurements is the wind. However, on the both measuring days the wind speed was less than five m/s^2.

9.3.1 Controlled walking test

In Figure 30, a curve with measured vertical accelerations from one of the controlled walking tests is shown. The measured accelerations from the controlled walking tests never exceed 0.5 m/s^2. The data shown in Figure 30 is measured in the quarter of the span in May with eight people walking on the bridge with a walking frequency of 1.9 Hz. The accelerations are highest when the group is passing the quarter of the bridge span, but when the group is passing midspan the accelerations are smaller.

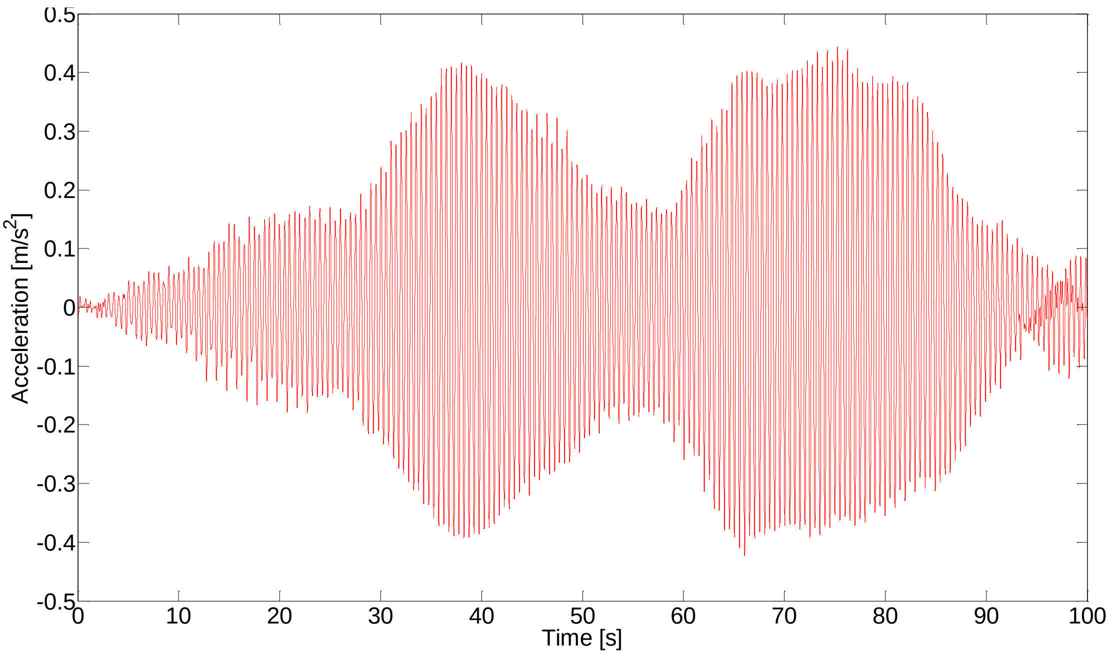


Figure 30 Measured vertical accelerations at Älvsbackabron when performing a controlled walking test with a walking frequency of 1.9 Hz.

One of the intentions of the controlled walking tests was to excite the bridge in lateral direction. Measured accelerations in lateral direction from a test with 1.5 Hz as walking frequency are shown in Figure 31. The maximum lateral acceleration is 0.037 m/s^2 and the accelerations are measured in the midspan.

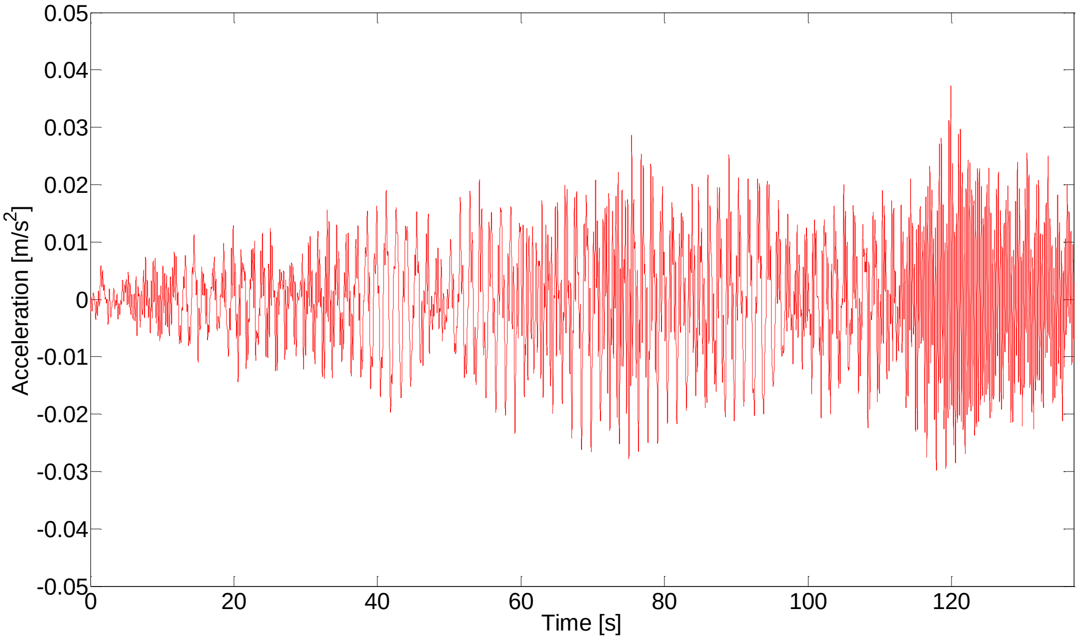


Figure 31 Measured lateral accelerations when performing a controlled walking test with a walking frequency of 1.5 Hz. The accelerations are measured in midspan.

During the both test occasions the cables of bridge started to sway from the applied forces in the controlled walking tests, but the oscillations decayed as soon as the group had left the bridge.

9.3.2 Jumping test

Results from the jumping tests will be presented below. The largest measured vertical acceleration from the jumping tests is 1.66 m/s^2, shown in Figure 32. The data from two additional jumping tests are shown in Figure 33 and in Figure 34. The three curves show the same tendency, the accelerations are increasing when the group is jumping in the middle of the bridge. After 30 seconds the group stop jumping and is standing still while the accelerations are decreasing.

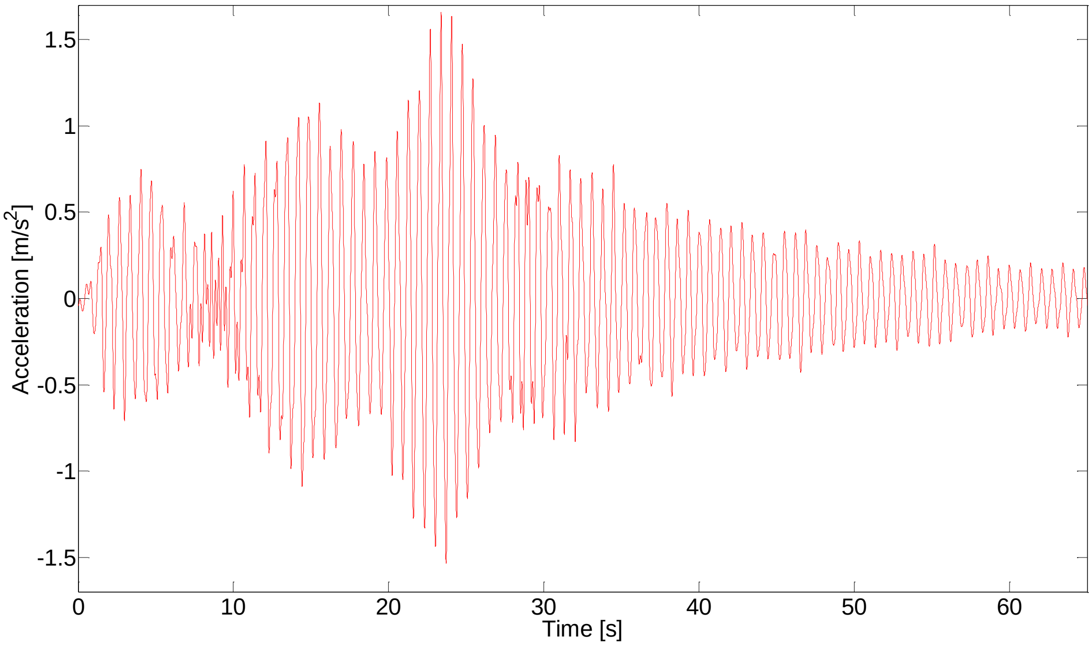


Figure 32 *Measured vertical accelerations at Älvsbackabron. This test resulted in the highest accelerations. The accelerations are measured in midspan during one of the tests performed in May with a jumping frequency of 1.4 Hz.*

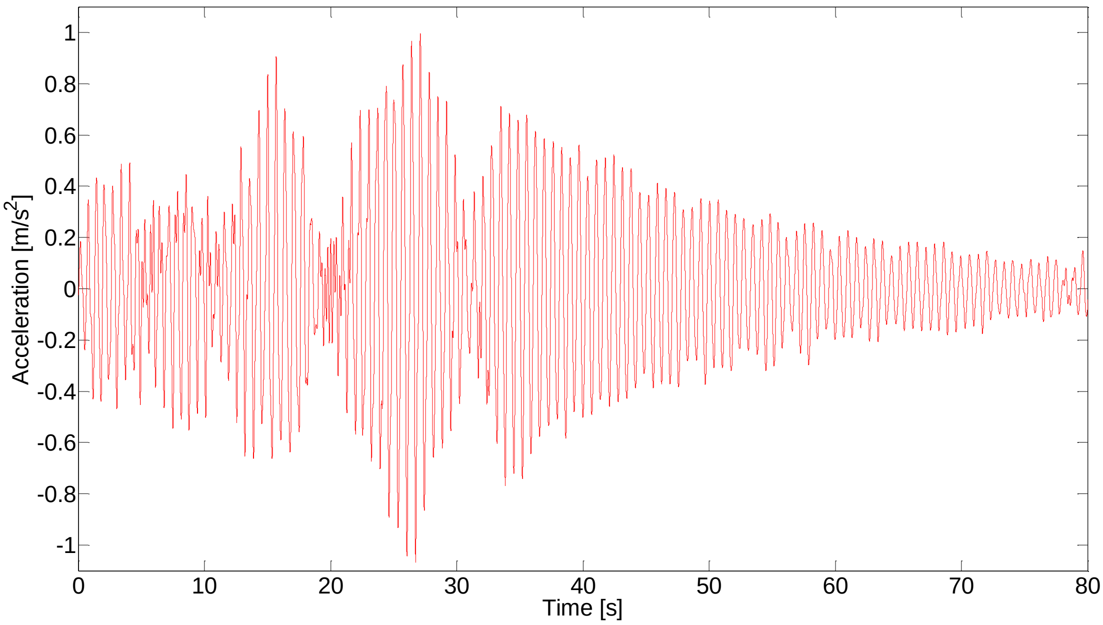


Figure 33 *Measured vertical accelerations at Älvsbackabron from one of the jumping tests. The accelerations are measured in midspan during one of the tests performed in March with a jumping frequency of 1.4 Hz. Different accelerometers are used in the tests performed in March, however, the results correspond well to the accelerations measured in May by another set of accelerometers.*

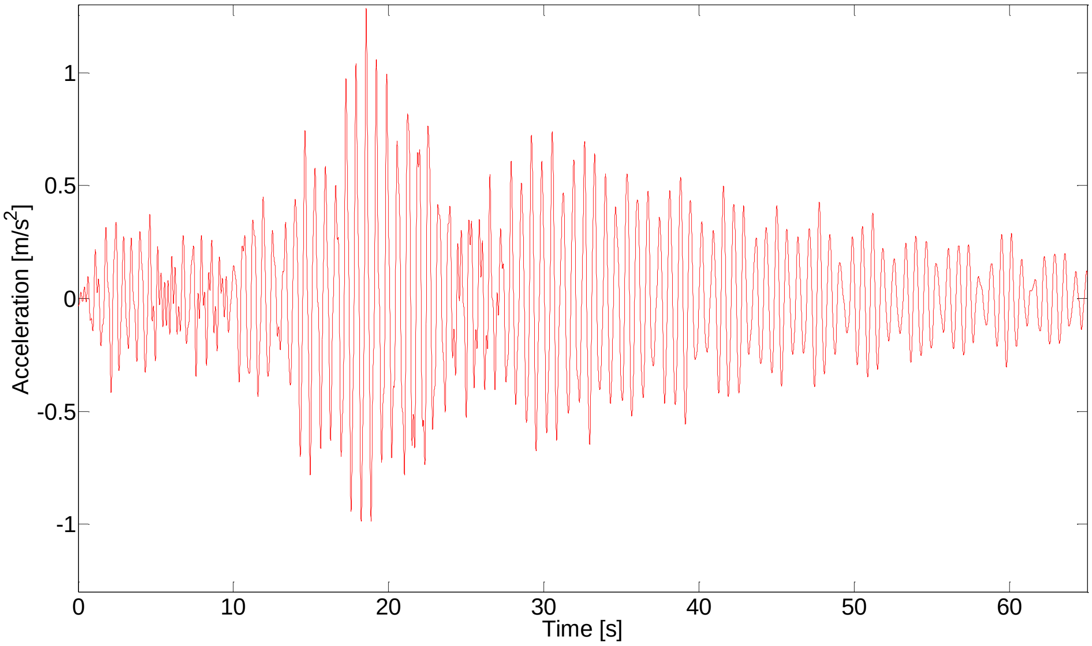


Figure 34 Measured vertical accelerations at Älvsbackabron from one of the jumping test. The accelerations are measured in midspan during one of the tests performed in May with a jumping frequency of 1.5 Hz.

An observation from the tests was the sway of the cables. During the jumping part of the jumping tests the cables started to oscillate. These oscillations continued for about one minute after the group had stopped jumping.

The results from the jumping tests are used to calculate a damping factor of the bridge. To calculate the damping factor curve fitting is performed in MATLAB. The MATLAB-code used to perform the curve fitting are attached in Appendix E. One of the fitted curves with decaying accelerations is shown in Figure 35. Taking results from all jumping tests into account the approximate damping factor from the jumping tests is estimated to 0.6%.

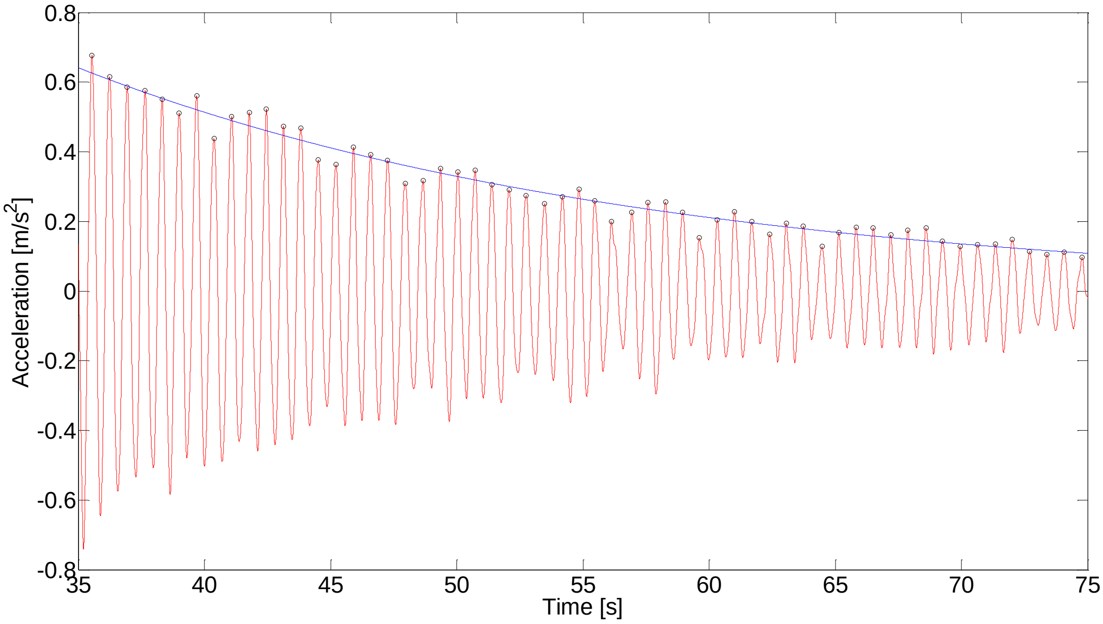


Figure 35 *Curve fitting of measured data to calculate the damping factor of the bridge. The fitted curve results in a damping of 0.6%.*

The ice skating jumps resulted in the highest measured lateral accelerations with a maximum acceleration of 0.14 m/s^2 in lateral direction. Figure 36 shows the resulting accelerations from one of the tests where the group was jumping sideways in the middle of the bridge for 30 seconds with a frequency of 1.4 Hz.

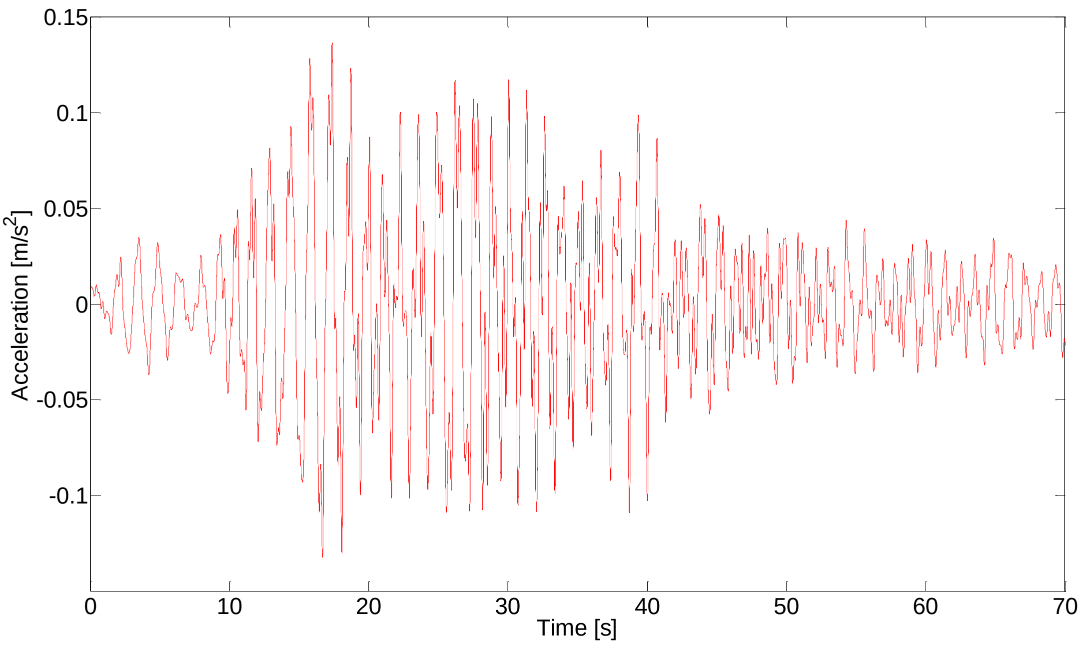


Figure 36 *Lateral accelerations from one of the ice skating jump tests performed in May. Accelerations are measured in the midspan.*

9.3.3 Heel impact test

Figure 37 shows the vertical accelerations from two heel impacts where the load is applied in midspan and the accelerations are measured in the quarter of the span. The maximum acceleration, measured both in midspan and in the quarter of the span, is approximately 0.1 m/s^2.

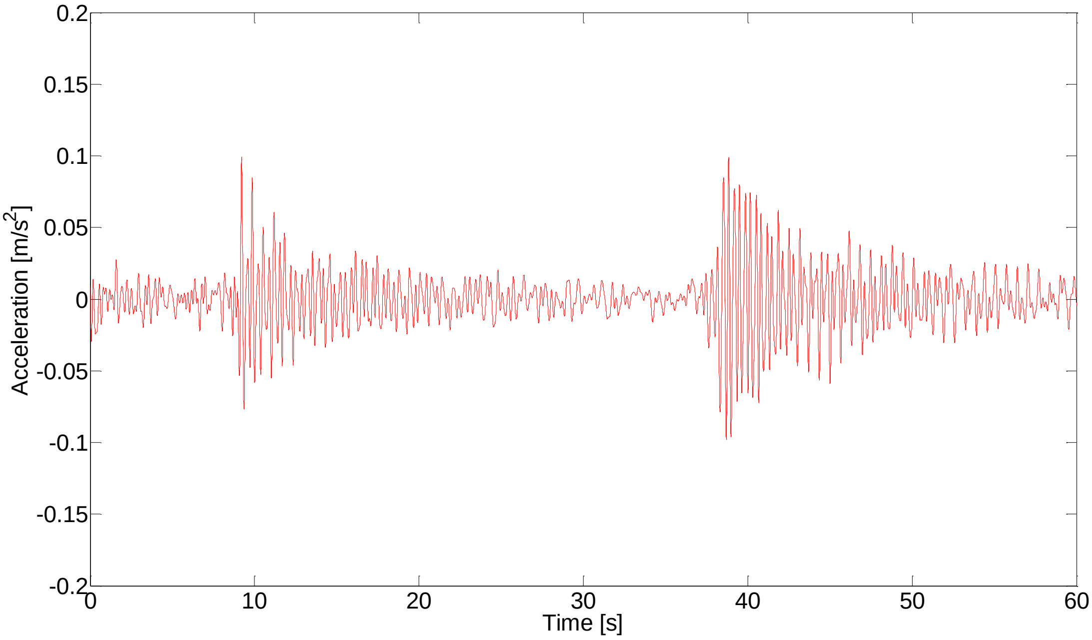


Figure 37 *Vertical accelerations from one heel impact test. The group applied the load in midspan and the result is measured in the quarter of the span. The test was performed in May.*

In Figure 38, the vertical accelerations from another heel impact test are shown. In this test the load is applied in the quarter of the span and the resulting accelerations are measured in the middle of the bridge.

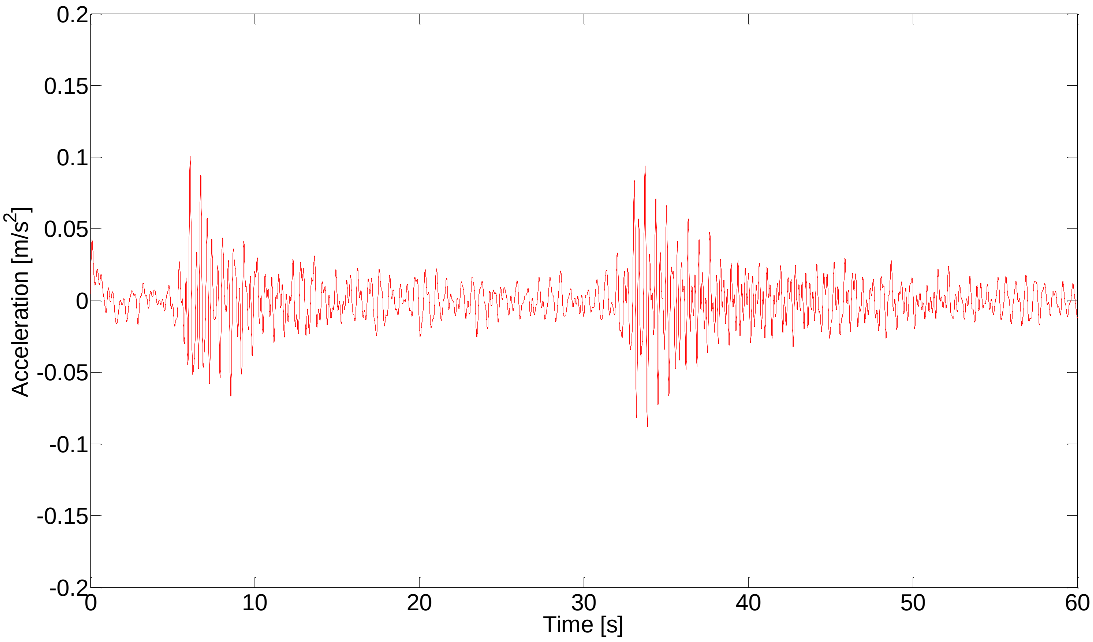


Figure 38 *Vertical accelerations from one heel impact test. The load from the group is applied in the quarter of the span and the response is measured in the middle. The test was performed in May.*

The reason the accelerations from the heel impact tests are not measured in the same position as the application point of the force is to avoid local irregularities. For example, all participants of the test may not stand completely still after the impact which results in additional local impacts. When performing these tests no persistent vibrations in the cables were observed.

The damping is calculated through curve fitting and Figure 39 shows the curve fitting for one of the heel impact tests. From all heel impact tests the damping factor is calculated to approximately 1.2% which is twice the damping calculated from the jumping tests.

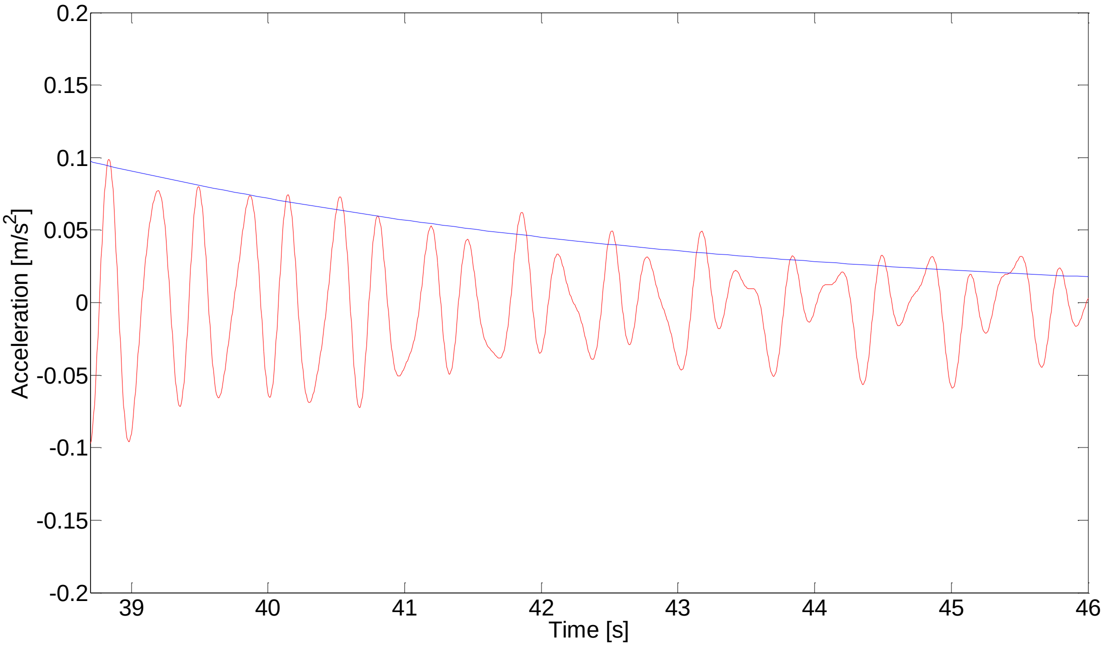


Figure 39 Curve fitting for one of the heel impact tests with a fitted curve with a damping factor of 1.2%.

9.3.4 Continuous measurements

The results from the continuous measurements at Älvsbackabron show that the vertical accelerations never exceed 0.2 m/s^2 and the lateral is about ten times smaller. In Figure 40, the vertical accelerations from one of the continuous measurements during a quarter of an hour measured in the quarter of the bridge span are shown. Figure 41 shows the corresponding lateral accelerations.

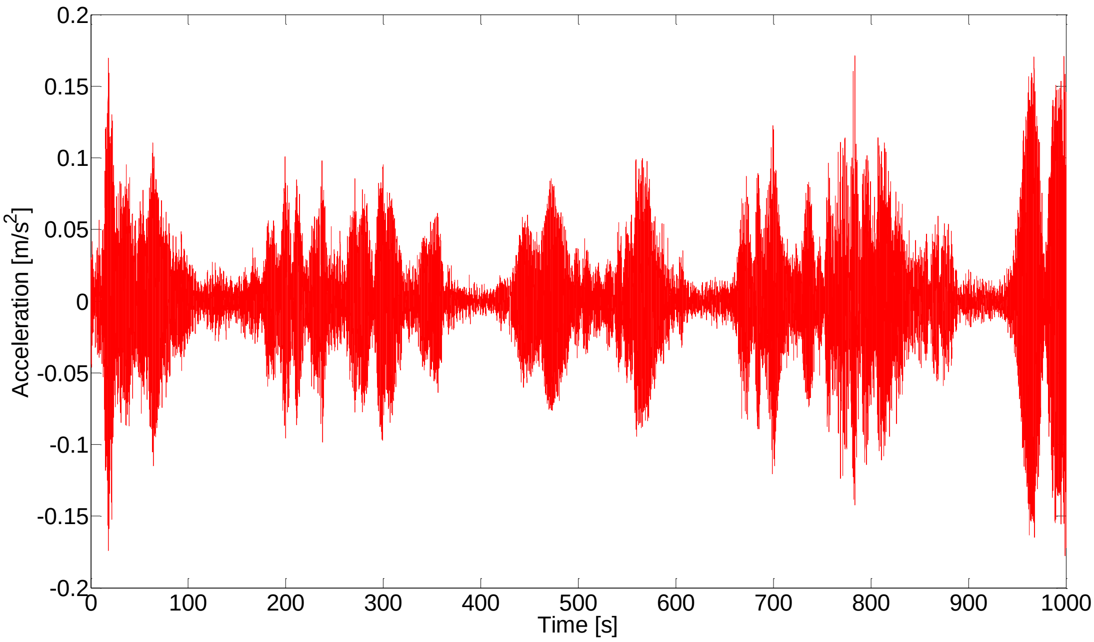


Figure 40 Vertical accelerations from one continuous measurement measured in the quarter of the span.

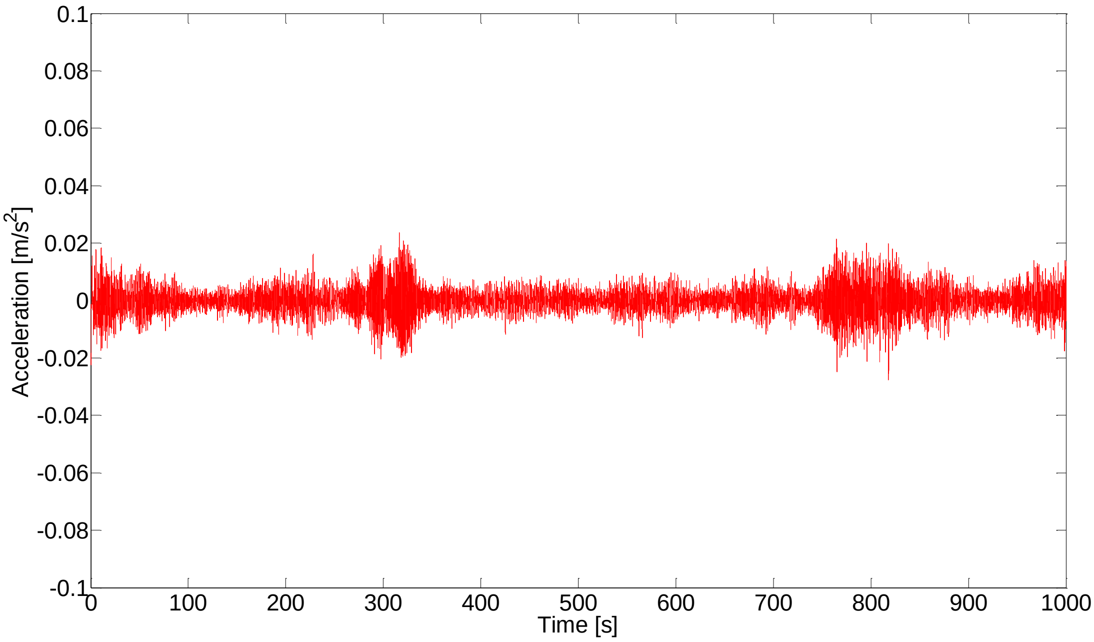


Figure 41 *Lateral accelerations from one of the continuous measurements, measured in the quarter of the span.*

During the continuous measurements pedestrians and runners crossed the bridge. Movement of the bridge deck from forces induced by other pedestrians was felt when standing still, however, when walking the movement of the bridge deck was not experienced as extensive. No significant sway of the cables was observed. These observations are the authors' personal experiences.

9.4 Simulations of the tests in Brigade/Plus

The simulations of the jumping tests and the heel impact test are modelled in the Master's thesis model based on material parameters given in Eurocode except for the damping factor which is the same value as calculated in corresponding test. The modelling of the forces are described in Section 8.3.

The resulting vertical accelerations from the jumping test simulated in the Master's thesis model according to Eurocode are shown in Figure 42 and in Figure 43 the results from the simulation of the heel impact test are presented as vertical accelerations of the bridge deck due to the impact force.

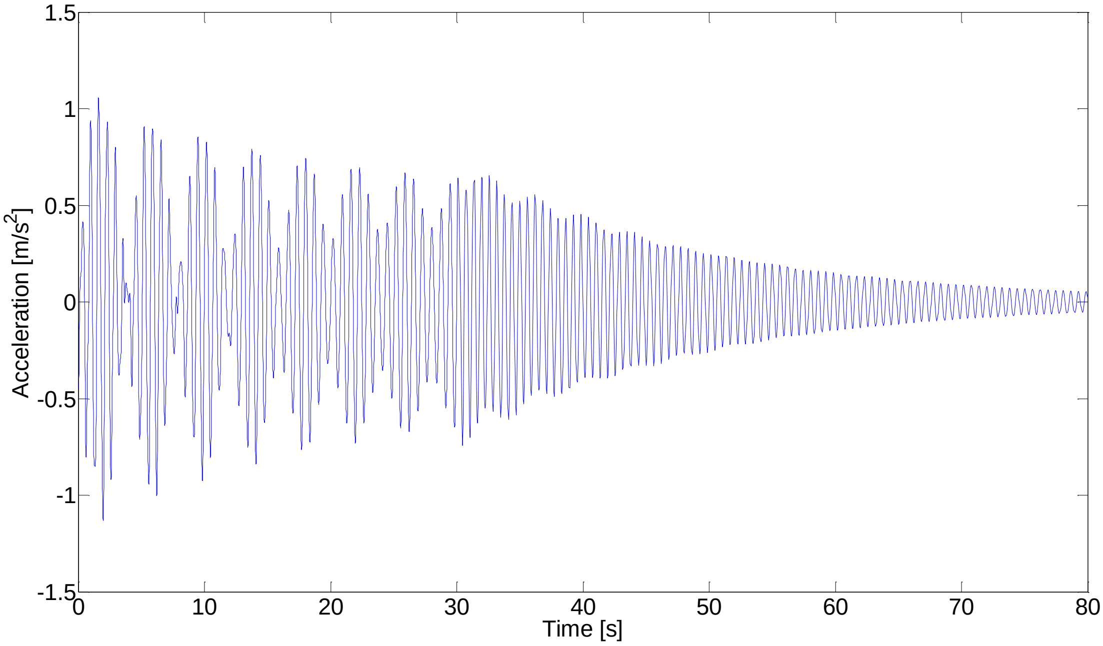


Figure 42 *Vertical accelerations from simulation of a jumping test in the Master's thesis model according to Eurocode. Damping factor of 0.6%.*

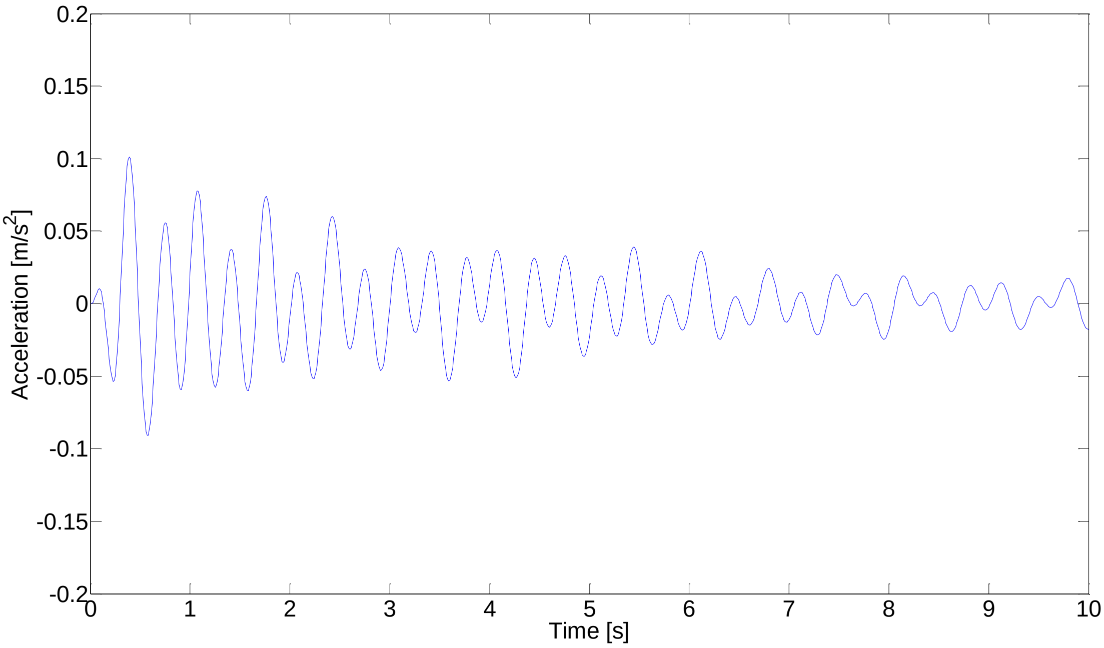


Figure 43 *Vertical accelerations from simulation of the heel impact test in the Master's thesis model according to Eurocode. Damping factor of 1.2%.*

10 Comparison

The comparison in this chapter is divided into two parts. The first part is a comparison between the different regulations and force models described in Chapter 3 and Chapter 4. The second part is regarding the case study of Älvsbackabron where the results from the two Master's thesis models are compared with each other and also with the measured results from Älvsbackabron.

10.1 Regulations and force models

The comfort criteria regarding the pedestrian induced vibrations presented in Chapter 3 all assign a limit for the vertical acceleration in the bridge deck. In Figure 44 the vertical limit acceleration for a frequency interval between 1 Hz and 5 Hz is presented. The peak acceleration limits given in both Eurocode 0 and BRO 2004 have a fixed value of 0.7 m/s^2 over the frequency interval. These two codes differ from the remaining codes which all have frequency dependent limits over the interval. Low limit accelerations for low frequencies takes into account the fact that people are more sensitive to low frequency vibrations. This is the case for the limits in the Danish standard Belastnings- og beregningsregler for vej- og stibroer and BS 5400 which both have increasing acceleration limits with increasing frequency. The main difference is that the limit in the Danish standard is stricter than in BS 5400.

In ISO 10137, however, the curve shows the reverse behaviour with a higher value of the limit acceleration for low frequencies. Besides, ISO 10137 also defines a curve with stricter limit accelerations for the design case with one person standing still in the midpoint of the bridge. This curve has the same tendency as the curve from ISO 10137 in Figure 44, but the values are half the values.

Notable is also that all curves, except the one from the Danish standard, have the same limit for a walking frequency of 2 Hz, namely 0.7 m/s^2. A walking frequency of 2 Hz is the most common walking frequency.

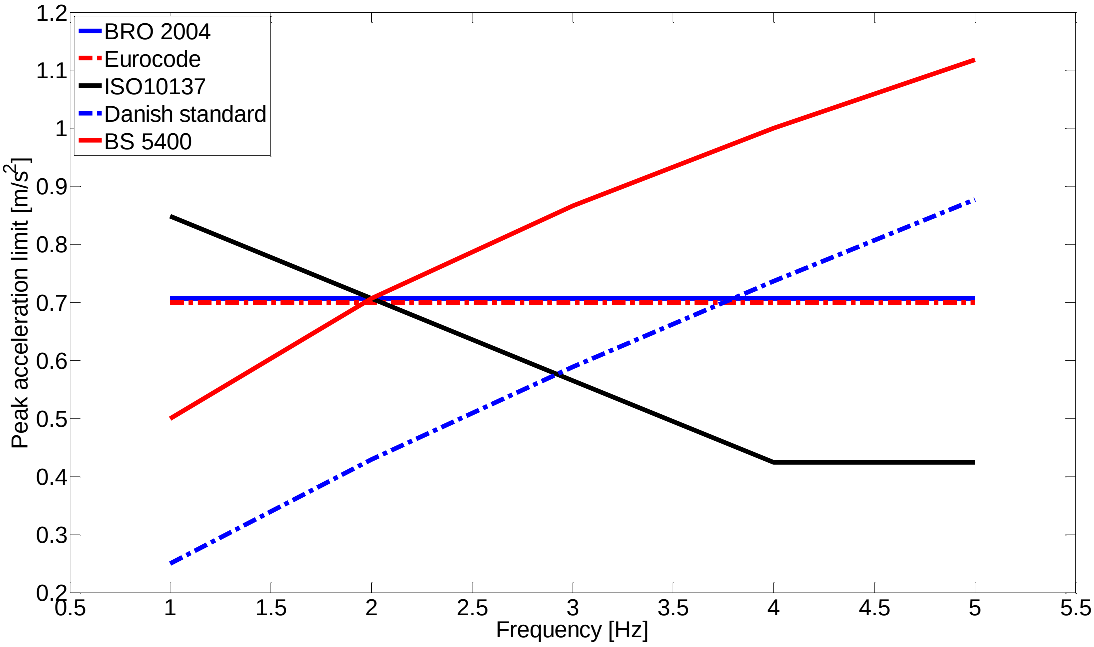


Figure 44 Peak acceleration limits for vertical accelerations presented in the different codes.

In the report by JRC and ECCC presented in Section 3.2.2, the limit acceleration is no longer frequency dependent, but is instead given by different comfort classes. Comfort class 1 describes the maximum comfort with a limit for vertical accelerations of 0.5 m/s^2. The value 0.7 m/s^2 belong to comfort class 2 which corresponds to medium degree of comfort. The upper limit for comfort class 2 is 1 m/s^2 and in relation to the limits presented in Figure 44 it is only BS 5400 that reaches this limit within the interval.

The peak acceleration limit curves for the lateral accelerations in the interval 1 to 5 Hz are presented in Figure 45. BRO 2004 do not specify any regulations regarding lateral acceleration and BS 5400 only state that special considerations regarding these accelerations have to be taken for bridges with low natural frequency.

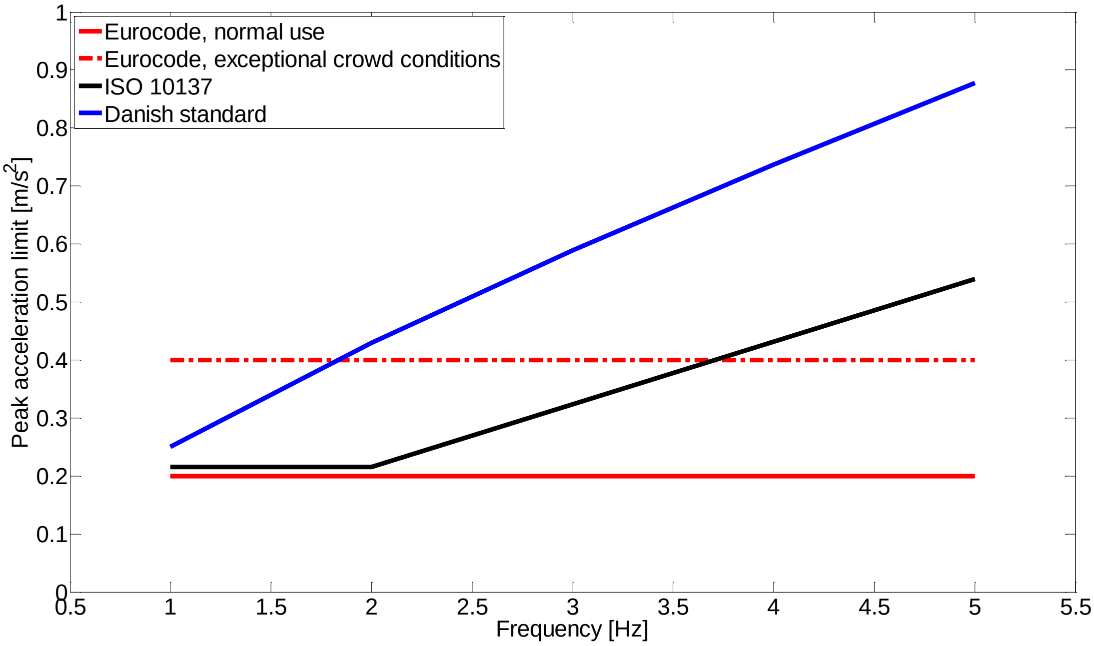


Figure 45 Peak acceleration limits for lateral accelerations presented in the different codes.

In Eurocode 0 two fixed limit values regarding lateral accelerations are given, one limit of 0.2 m/s^2 which is relevant for the daily use of the bridge and a higher limit of 0.4 m/s^2 for exceptional crowd conditions. The limit of 0.2 m/s^2 is the strictest value for the lateral accelerations.

The lateral limit in ISO 10137 is fixed up to 2 Hz and then frequency dependent. The limit given in Danish standard is also frequency dependent. For the lowest frequency all the limits are almost the same, about 0.2 m/s^2, except the higher Eurocode limit, but the limits differ for higher frequencies.

The limits in Figure 45 can be compared with the lateral limit accelerations presented by JRC and ECCC. For maximum comfort, comfort class 1, the lateral acceleration should not exceed 0.1 m/s^2. For medium comfort the acceleration should be within the span of 0.1-0.3 m/s^2. These values are stricter compared to the limits presented in Figure 45.

The force models given in BRO 2004 and the Danish standard resemble each other with a periodic force. The differences are mainly the magnitude of the force amplitude and the movement of the pedestrian force in the Danish standard. None of the codes present force models for the lateral force.

The vertical force model given in BRO 2004 takes the size of the bridge and hence the possible amount of pedestrians into consideration. Included is also the fact that the force might cause resonance for frequencies below 2.5 Hz why a higher force amplitude is used for frequencies below 2.5 Hz. These aspects are not considered in the Danish standard. An additional difference between the force models is the amplitude. The amplitude is smaller in the Danish standard with a value of 360 N. This fact should be put in relation to the strict limits for vertical accelerations given in the Danish standard. No models regarding the lateral force are given in the two codes.

The Eurocode and Swedish national annex provides no force model for the pedestrian force other than for the design case of a simply supported bridge. Instead, the ISO

10137 standard can be used. A difference with this model compared to the two previously described is that this model includes the static force from the pedestrians. The ISO 10137 force model also takes the contribution of higher harmonics from the pedestrian force into account by introducing a phase shift. Thus, the force model in ISO 10137 is not harmonic as the other presented forces. ISO 10137 also takes the number of pedestrians into account and adds a coordination factor which describes an equivalent number of people who will walk with the same pace. The ISO 10137 standard also includes a force model for the lateral force. This model is the same as the model for the vertical force, but with different values of the coefficients.

10.2 Master's thesis models

In this second part of the comparison, the Master's thesis models according to BRO 2004 and Eurocode will be compared. The two main differences between the models are the material parameters and the force models.

10.2.1 Material parameters

The effect from the different material parameters on the two Master's thesis models are studied by comparing the natural frequencies of the two models. The parameters that differ are presented in Table 19 and in Appendix A a complete list of all material properties is presented. The largest differences between the two models are the density of the timber materials and the given damping factors.

Table 19 Comparison of material properties from BRO 2004 and Eurocode 5.

Comparison element	BRO 2004	Eurocode 5
Young's modulus glulam	13 GPa	13.7 GPa
Young's modulus cables	200 GPa	210 GPa
Density glulam	600 kg/m²	430 kg/m²
Density timber K24	600 kg/m²	430 kg/m²
Damping factor	0.6%	1.5%

The differences between the material parameters affect the results in the models. In Table 20 the natural frequencies for the both models are shown. The natural frequencies are higher in the Master's thesis model according to Eurocode. The difference in damping factor does not affect the natural frequencies, but only the magnitude of the accelerations.

Table 20 Comparison of the first ten natural frequencies calculated for the two Master's thesis models.

Description of mode shape	Natural frequencies from Master's thesis models according to BRO 2004 [Hz]	Natural frequencies from Master's thesis models according to Eurocode [Hz]
Lateral movement of bridge deck	0.620	0.746
Vertical movement of bridge deck	1.386	1.648
Lateral movement of pylon	1.390	1.539
Lateral movement of bridge deck	1.441	1.732
Lateral movement of pylon	1.479	1.644
Vertical movement of bridge deck	1.899	2.285
Torsional movement of bridge deck	2.350	2.714
Lateral movement of bridge deck	2.559	3.057
Longitudinal movement of pylons	2.760	3.244
Longitudinal movement of pylons	2.786	3.274

10.2.2 Force models

Since no force model is given in Eurocode for this bridge type, the comparisons are between the force model in BRO 2004 and the simplified force model from ISO 10137 instead. Both force models are described in Section 7.2.

As described in Section 3.3, the ISO 10137 standard states different design situations which are decisive for the force. The worst case scenario is pedestrian streams significantly larger than fifteen people, why in this comparison a group of fifty pedestrians is chosen for the calculations. The force amplitude from the simplified ISO 10137 model is 1700 N while in BRO 2004 it is 1082 N. In the ISO 10137 force model a group of fifty pedestrians is chosen and within this group $\sqrt{50}$ pedestrians are walking with the same pace.

Figure 46 shows the vertical acceleration response from the two load models acting on the Master's thesis model according to BRO 2004. The frequency of the force is sweeping from 0-5 Hz, the peaks indicates the resonance frequencies of the bridge deck.

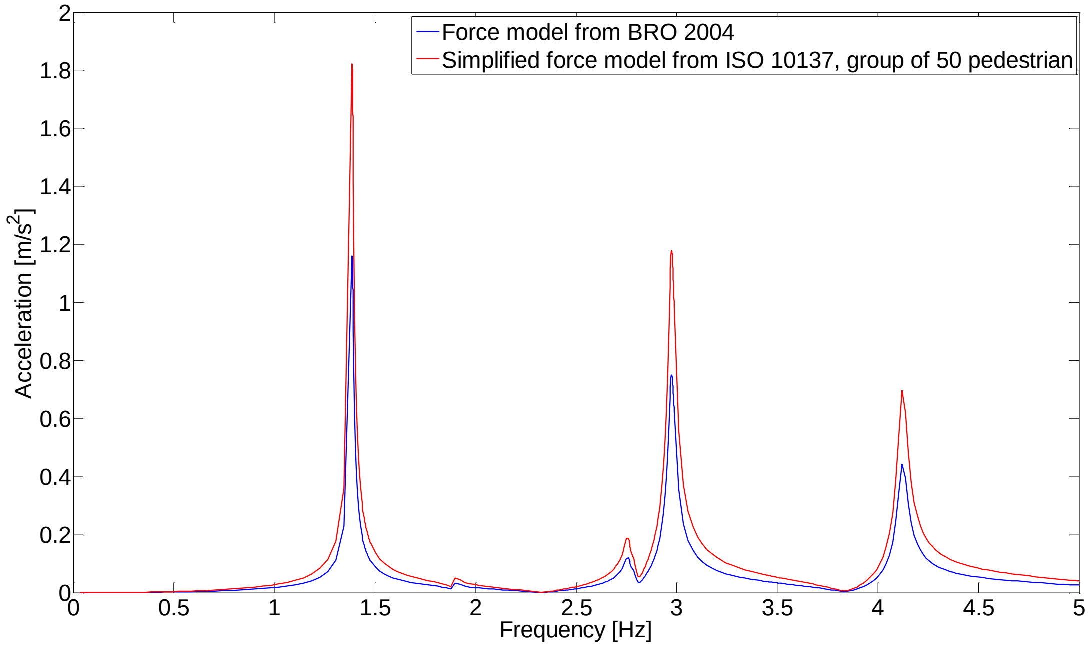


Figure 46 Vertical acceleration response due to the force models in BRO 2004 and the simplified force model in ISO 10137 with a group of fifty pedestrians applied on the Master's thesis model according to BRO 2004.

Naturally, the force from the ISO 10137 model gives higher vertical accelerations because the magnitude of the force is higher. Figure 47 shows the vertical acceleration response from the two force models applied in the Master's thesis model according to Eurocode.

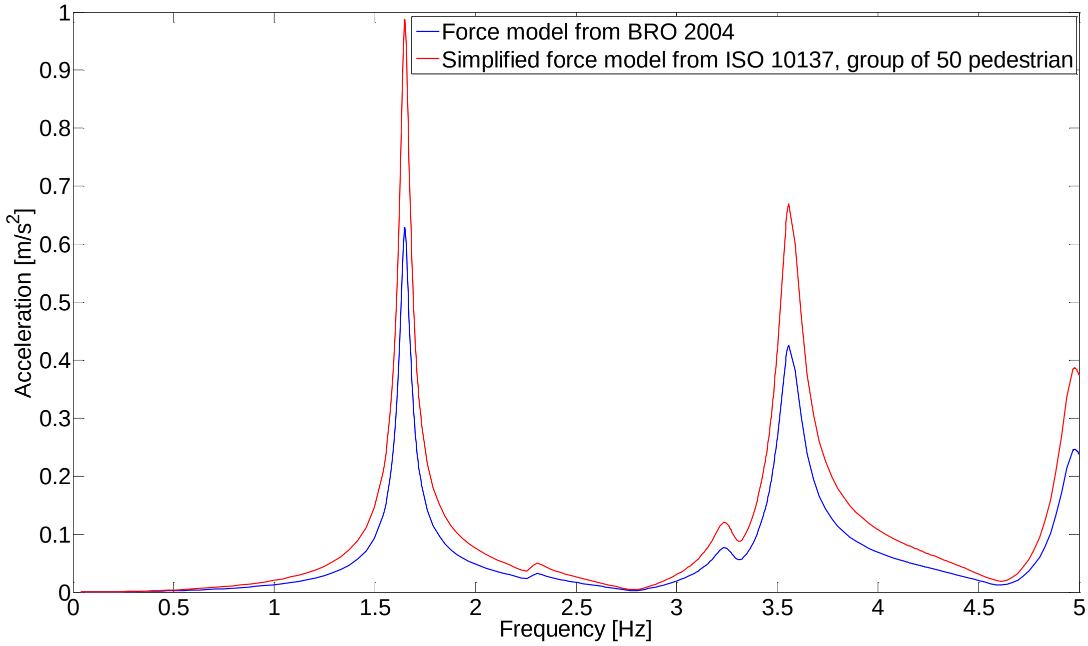


Figure 47 Vertical acceleration response due to the force models in BRO 2004 and the simplified force model in ISO 10137 with a group of fifty pedestrians applied on the Master's thesis model according to Eurocode.

Also in this figure the accelerations due to the ISO 10137 force are higher, but the magnitudes of the accelerations differ between the two models. The Master's thesis model according to Eurocode gives a lower maximum value of the vertical acceleration due to the fact that the damping factor is higher.

For further comparison between the models, the same force is applied in the both Master's thesis models, namely the force model from BRO 2004 with a force magnitude of 1082 N. The results are shown in Figure 48, where it clearly can be seen that the Master's thesis model according to BRO 2004 results in higher vertical accelerations and lower natural frequencies than the Master's thesis model according to Eurocode. The reason for this is the difference in the material parameters. According to Hallgren[6] the density of timber and glulam given in BRO 2004 is too high, but is assumed to include non-structural mass, while the value given in Eurocode 5 are too low and all extra mass have to be considered separately.

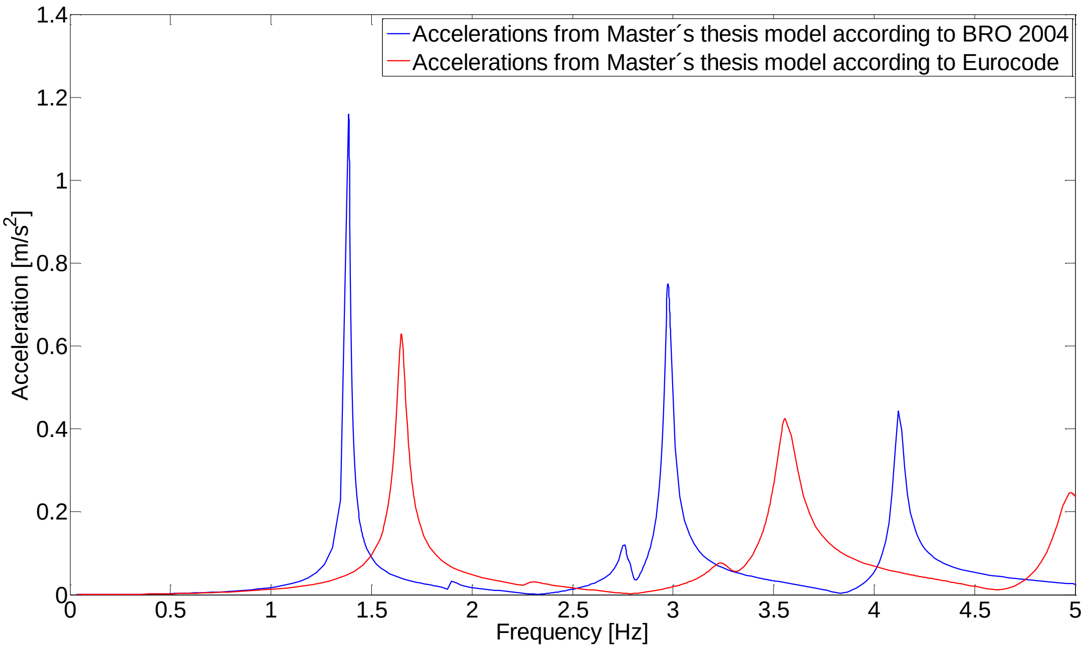


Figure 48 Vertical accelerations from the force model in BRO 2004 applied on both Master's thesis models.

The most relevant comparison is between the resulting accelerations from the Master's thesis model according to BRO 2004 and resulting accelerations from Master's thesis model according to Eurocode. The accelerations are achieved by applying the corresponding force model on the two Master's thesis models. The resulting vertical accelerations are shown in Figure 49. It can be seen that even though the force amplitude from the simplified force model in ISO 10137 is higher than the force amplitude in BRO 2004 the resulting accelerations is higher in the Master's thesis model according to BRO 2004. This is due to the fact that the damping factor is higher according to Eurocode 5. The different natural frequencies from the two models can also be seen in Figure 49.

[6] Thomas Hallgren, structural engineer COWI AB, meeting March 21:th 2012

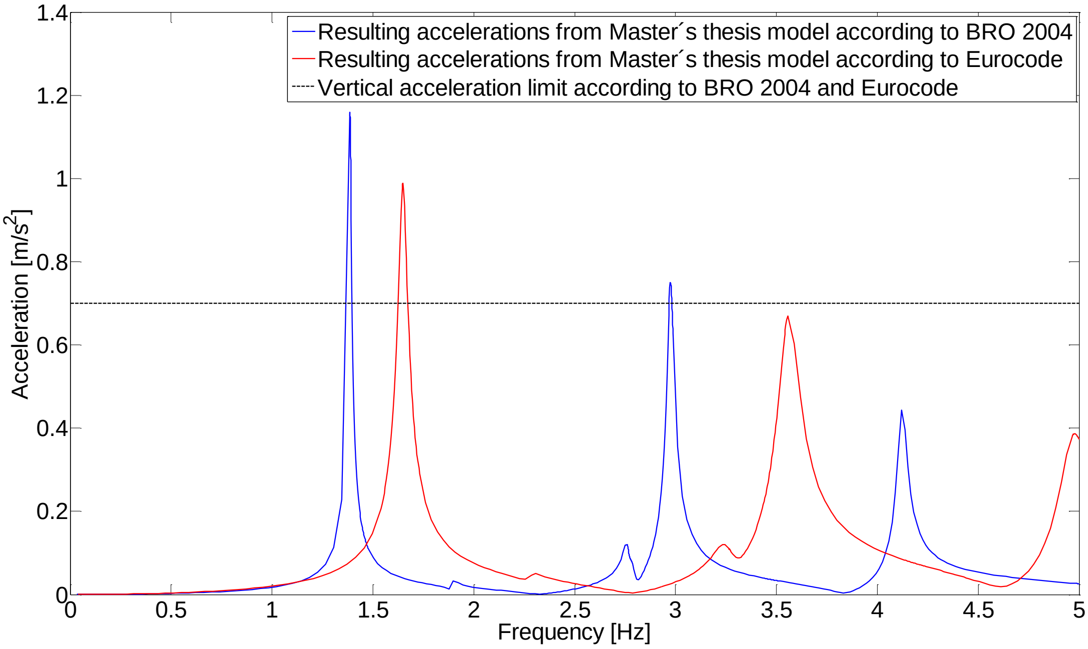


Figure 49 *Vertical acceleration response from the force model in BRO 2004 acting on Master's thesis model according to BRO 2004 and force model from ISO 10137 with a group of fifty pedestrians acting on the Master's thesis model according to Eurocode.*

The vertical acceleration limit is exceeded in both Master's thesis models. The force model given in BRO 2004 only depends on the size of the bridge why it cannot be varied. The force model in ISO 10137 depends on the number of pedestrians walking on the bridge at the same time. In all previous calculations a group of 50 pedestrians have been assumed, but the results exceed the limit value. To fulfil the vertical limit twenty pedestrians can be used in the simplified ISO 10137 force model. Resulting vertical accelerations from the simplified ISO 10137 force model with twenty walking pedestrians are shown in Figure 50.

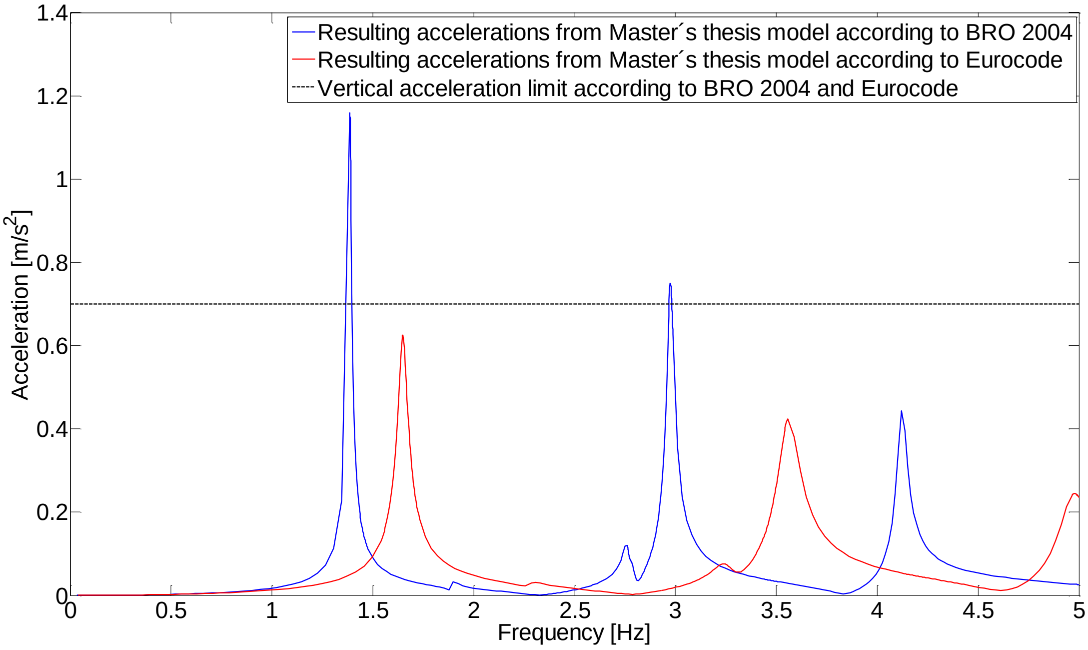


Figure 50 Vertical accelerations from the force model in ISO 10137 with a group of twenty pedestrians acting on the Master's thesis model according to Eurocode and the force model in BRO 2004 acting on the Master's thesis model according to BRO 2004.

If the lateral force in ISO 10137 with a group of twenty pedestrians is applied in the Master's thesis model according to Eurocode the lateral acceleration limit is exceeded.

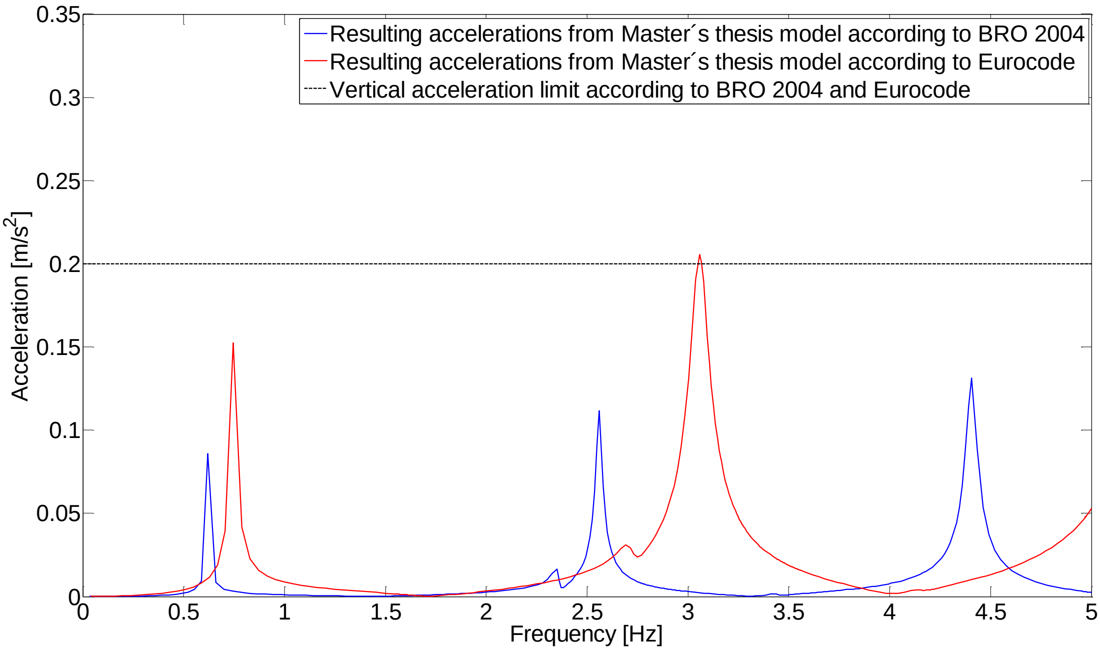


Figure 51 Lateral accelerations from the force model in ISO 10137 with a group of twenty pedestrians acting on the Master's thesis model according to Eurocode and the force model in BRO 2004 acting on the Master's thesis model according to BRO 2004.

10.3 Measurements from Älvsbackabron and simulations

In this section, the accelerations measured at Älvsbackabron are compared with the results from the simulations of the tests in Brigade/Plus. The compared tests are the jumping test and the heel impact test. In Figure 52, a comparison between the measured vertical acceleration from one of the jumping tests at Älvsbackabron and corresponding simulation in the Master's thesis model according to Eurocode is shown. In the simulation, a damping factor of 0.6% is used, which is the same as the calculated value from the measurements.

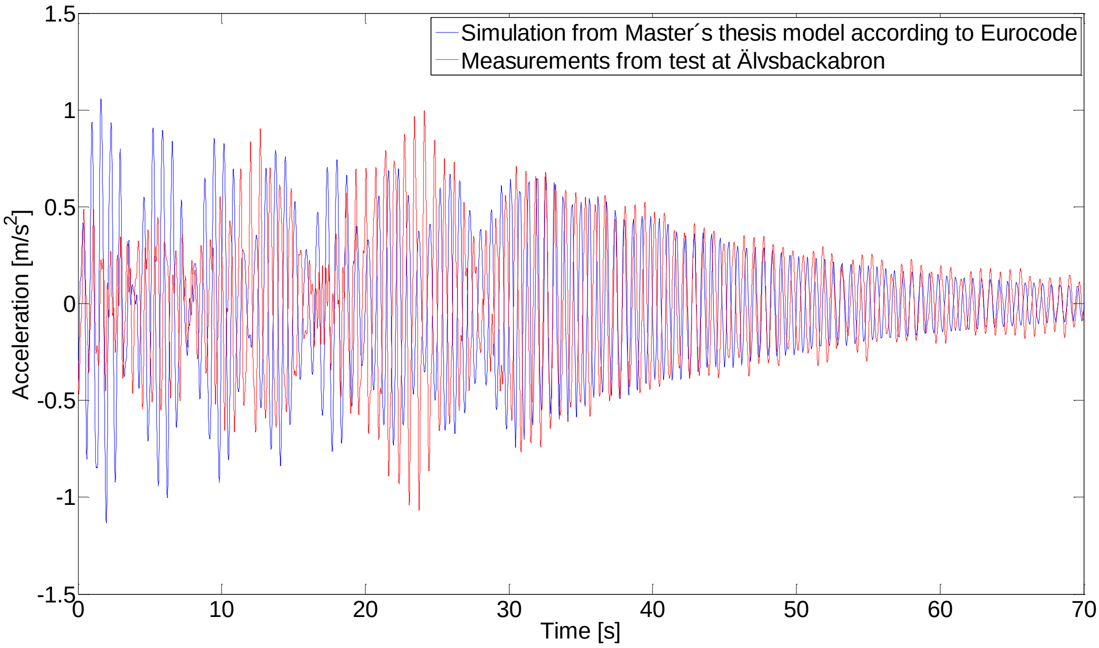


Figure 52 Comparison of the vertical accelerations from one jumping test performed at Älvsbackabron in March and from the simulation of the jumping test in Master's thesis model according to Eurocode except the damping factor which is 0.6%, same value as calculated from jumping tests.

The measured and simulated acceleration shows quite good correlation, especially when the energy of the system is dissipating. This is shown more clearly in Figure 53 where the accelerations are shown from when the jumping group has stopped jumping.

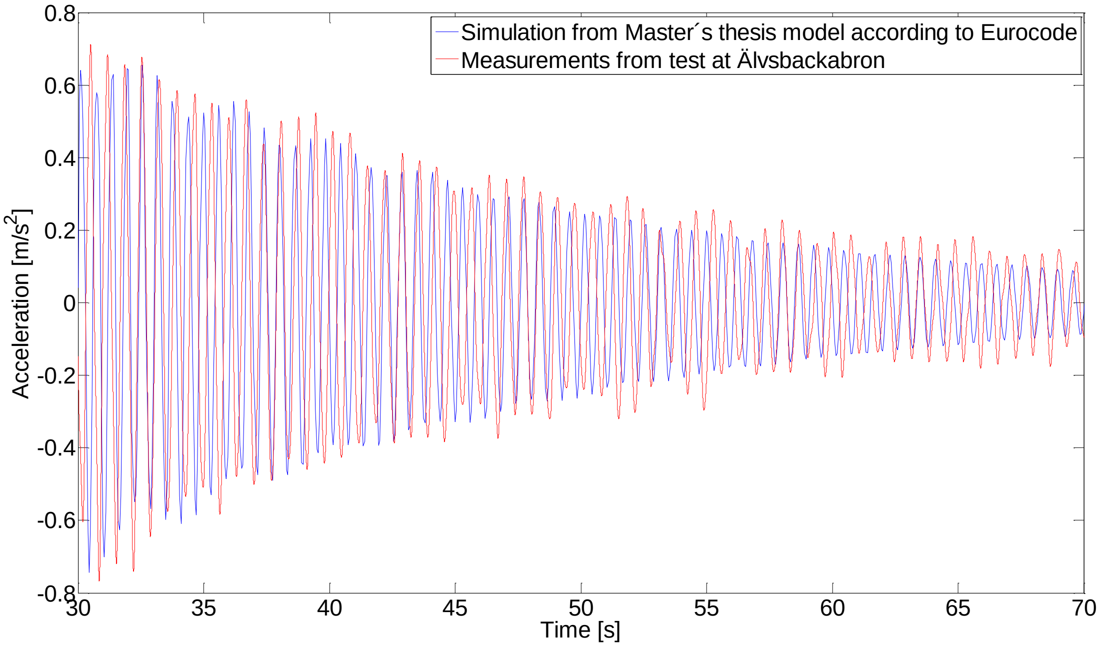


Figure 53 *Comparison of measured response of Älvsbackabron and simulations from Master's thesis model according to Eurocode after the group has stopped jumping.*

Furthermore, the simulated and measured accelerations from the heel impact test also show good correlation. In Figure 54 the comparison is showed for the first ten seconds after one impact. In the simulation of the heel impact test, a damping factor of 1.2% is used, the same as calculated from the measured heel impact test.

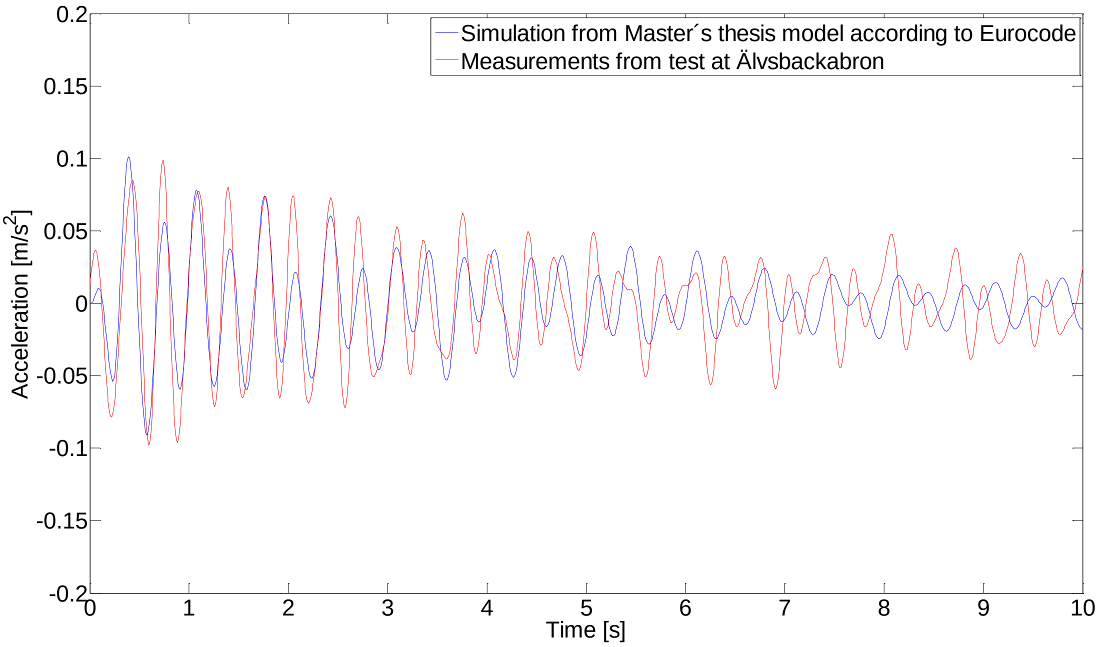


Figure 54 *Comparison of measured vertical accelerations from one heel impact test at Älvsbackabron and corresponding simulation in the Master's thesis model according to Eurocode except the damping factor which is 1.2%, same value as calculated from heel impact tests.*

To estimate the real natural frequencies of Älvsbackabron the measured accelerations are transformed into the frequency domain with Fast Fourier Transformation, FFT. The transformation process in MATLAB is described in Appendix E.

The FFT-curve from one of the jumping tests is shown in Figure 55, where the first vertical natural frequency is found to be 1.45 Hz.

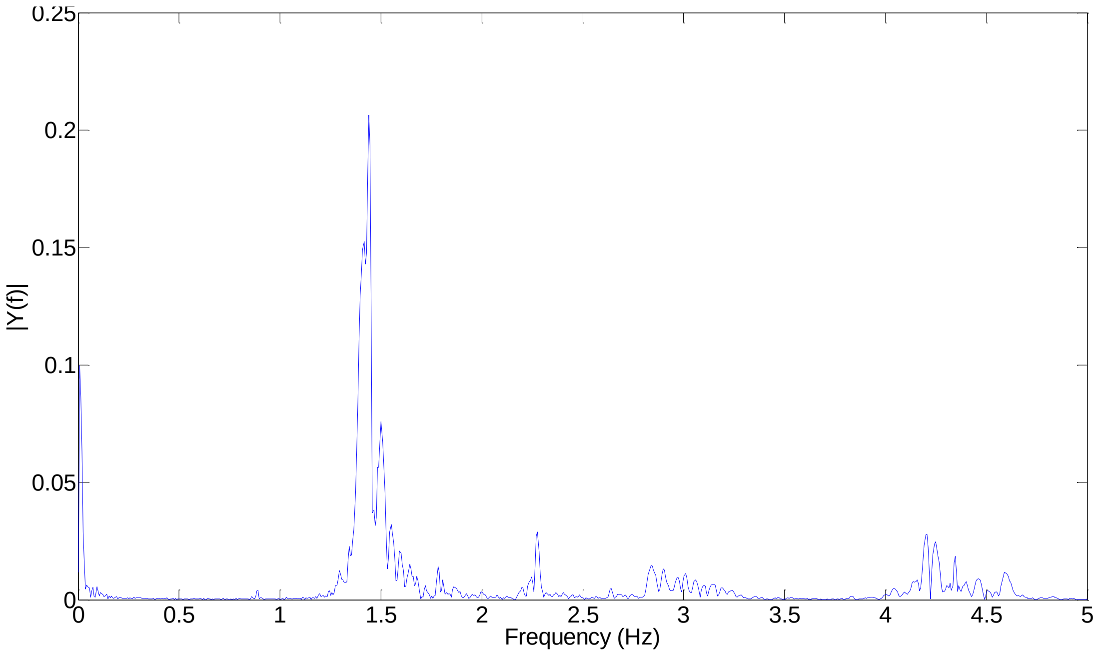


Figure 55 FFT-curve from one of jumping tests measured in the middle when jumping in midspan with a frequency of 1.4 Hz.

In Figure 56 a second FFT-curve from Älvsbackabron is shown. This figure shows a natural frequency of 2.01 Hz which is close to the theoretical second vertical natural frequency. The curve is based on accelerations measured in the quarter of the span from a controlled walking test with a walking frequency of 1.9 Hz.

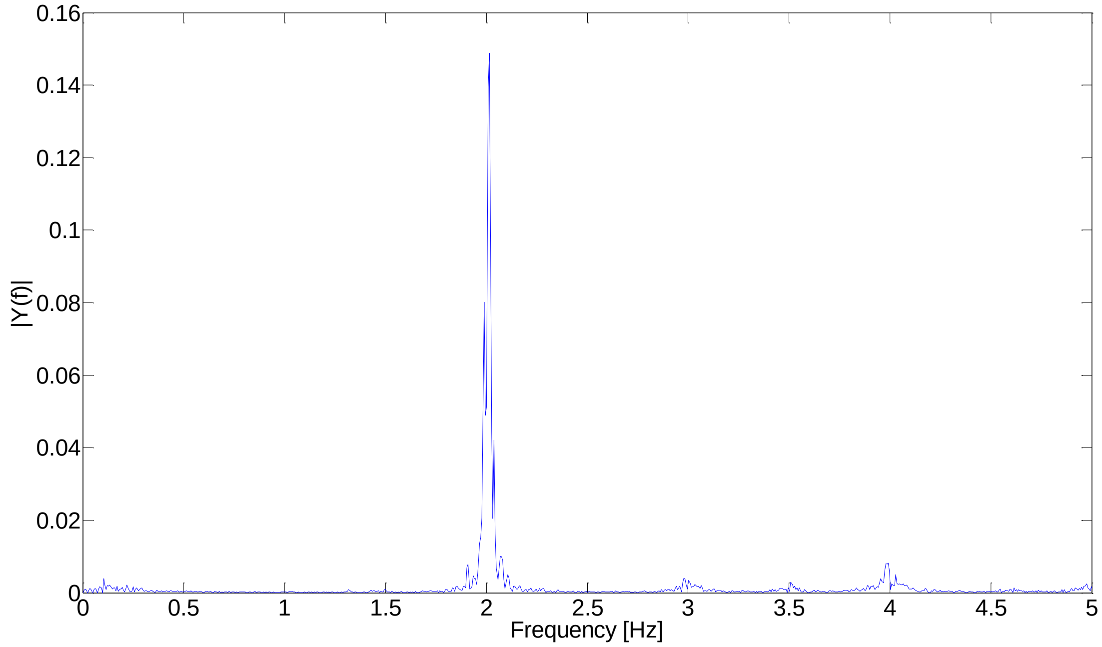


Figure 56 FFT-curve from one of the controlled walking tests at Älvsbackabron. The vertical accelerations are measured in the quarter of the span and the walking frequency is 1.9 Hz.

Based on these two FFT-curves the first two natural frequencies of the bridge seem to be in the range between theoretical values from the Master's thesis models. A compilation of the frequencies are shown in Table 21.

Table 21 Comparison of the natural frequencies measured at Älvsbackabron and resulting from the Master's thesis model according to BRO 2004 and Master's thesis model according to Eurocode.

	Master's thesis model according to BRO 2004 [Hz]	Measured frequencies at Älvsbackabron [Hz]	Master's thesis model according to Eurocode [Hz]
First vertical	1.386	1.45	1.648
Second vertical	1.899	2.01	2.286

From measured values from the heel impact tests the damping factor is calculated to be 1.2%, which is between the value of 0.6% given in BRO 2004 and the value 1.5% given in Eurocode 5. In Figure 57, the effect of the damping factor is shown. The load from BRO 2004 has been applied in the Master's thesis model according to BRO 2004 with two different damping factors. The resulting accelerations from the model with the higher damping factor fulfil the acceleration limit.

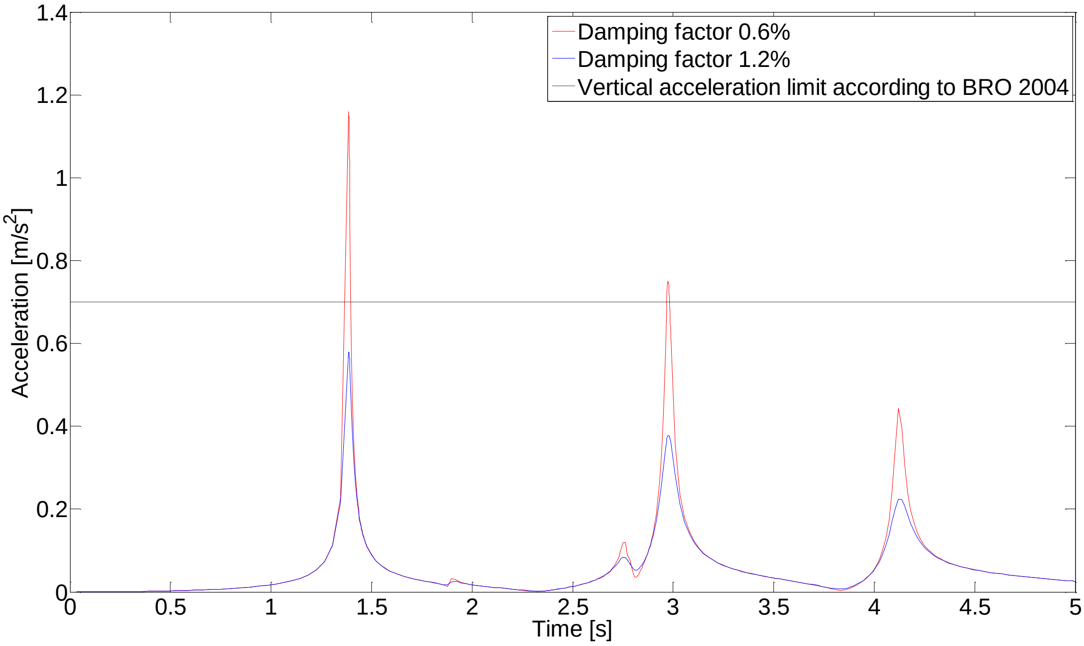


Figure 57 Illustration of the effect of different damping factors. The Master's thesis model according to BRO 2004 is modelled with 0.6% damping factor and 1.2% damping factor. The applied force is the force from BRO 2004.

11 Discussion

In this chapter the results and the comparisons are discussed by the authors. The discussion are based on facts presented in the chapters regarding regulations and force models and also on own assumptions and the measured results at Älvsbackabron.

11.1 Regulations and force models

The main difference between the limits for vertical accelerations presented by different codes as shown in Section 10.1 is, if the limit is frequency dependent or not. The limits presented in the Danish standard and BS 5400 are frequency dependent limits that assign higher limit values for higher frequencies. An opposite behaviour is shown by the vertical limit presented in ISO 10137 in which a high limit is assigned for lower frequencies and a low fixed limit for frequencies above 4 Hz. The curve is interpreted by the authors to reflect the behaviour of vertical accelerations in the bridge deck, where the lowest frequencies experience the highest accelerations. Though, the limit of 0.4 m/s^2 for frequencies above 4 Hz is low compared to the other limits. It should be kept in mind that the ISO 10137 standard not only considers dynamic design of footbridges, but also dynamic design of buildings. Thus the standard is not a refined bridge standard, but a vibration standard.

In the ISO 10137 standard different acceleration limits are given for different design situations. This principle is further developed by the Joint Research Centre and European Convention for Constructional Steelworks since traffic class, occurrence and suitable comfort class have to be chosen in each individual project. The method of determining a number of design situations with different comfort criteria seems reasonable since it results in flexible limits which can be easily adapted to the specific design situation. However, this method might be more demanding for the client and structural engineer since relevant design situations and their occurrences have to be determined in advance for each project. Good communication between the structural engineer and the client might be necessary to assign the design situations for each project. Also, to set suitable design situations with relevant limits and traffic classes some experience is needed.

The lateral limits are missing in BRO 2004 and BS 5400, but exists in Eurocode 0 as two fixed values depending on the amount of pedestrians crossing the bridge. In the ISO 10137 standard, a limit with a constant value of 0.2 m/s^2 is used for the frequencies up to 2 Hz and then the acceleration limit value increases. Also for lateral vibrations the method with design situations seems like a reasonable method for assessing suitable limit vibrations.

In Section 2.2 some additional limit values for vertical and lateral accelerations are described. A suggested vertical limit of 0.13 m/s^2 based on different perception levels of vibrations. This value is very low compared to the values given in the studied codes and standards. A high acceleration limit based on a full scale bridge test is presented for the lateral accelerations. A suggested limit is 1.35 m/s^2 which is much higher than lateral all limits presented in the studied codes and standards. These two limit values are both based on people's perception of vibration and the distribution of limit values show the difficulty in estimating a suitable acceleration limit.

The vertical force models presented in BRO 2004, the Danish standard and BS 5400 are all harmonic forces with different amplitudes. The force model given in the ISO

10137 standard differ from the others since it takes higher harmonics into account by introducing a phase shift and the static weight of pedestrians. Thus, the force is not harmonic, but instead a time dependent periodic force. The intention of including the static weight and the phase shift is to characterise the nature of the force from a pedestrian.

There are other force models presented in Section 4.6 which also aim to describe the nature of the force by including irregularities of the pedestrian force and the synchronization phenomena, also called lock-in effect. By doing this, a more accurate prediction of the forces induced by pedestrians can be achieved and perhaps more realistic load models can be established. One downside with the non-harmonic force models compared to the harmonic force models in the codes is that these models are more difficult to simulate in a computer model and may take longer time to calculate since transient analysis are needed. An additional disadvantage with the force models are the bridge specific coefficients that are needed. These are supposed to be established in an experimental way for the specific bridge which is not possible in design projects.

In the design phase it is often desirable to optimize the design process to minimize the required time and costs. One way to do this is to use force models that are suited for design with computer software.

Regarding the force models in lateral direction it is only the ISO 10137 standard that assigns a model for this force. Still, it is an important force to take into consideration when designing lightweight bridges. With the improvement and development of the Eurocodes this subject will hopefully have a greater significance and force models for both vertical and lateral pedestrian induced forces are hopefully included.

11.2 Case study of Älvsbackabron

One of the purposes of measuring accelerations at Älvsbackabron is to investigate if the measured accelerations exceed the limits or if they fulfil the design limits. The measurements show that the vertical acceleration limit in Eurocode 0, 0.7 m/s^2, is exceeded during the jumping test where the maximum measured vertical acceleration is 1.6 m/s^2. This value might seem high, but the jumping tests are considered to be extreme loading situations with large forces pulsating with the frequency close to the natural frequency of the bridge. The results from the controlled walking tests, which are considered to symbolise a possible loading situation, never exceeded 0.5 m/s^2 and the vertical accelerations from the continuous measurements never reached above 0.2 m/s^2. These values show that Älvsbackabron fulfils the design requirements regarding vertical vibrations presented in both Eurocode 0 and BRO 2004.

In the case study the vertical force model in the ISO 10137 standard is modelled as a harmonic load applied in the Master's thesis models. The simplification is described in Section 7.2 and the main reason for the simplification is to be able to calculate in a frequency domain. In the simplified force model all the harmonics are assumed to be in phase with each other and to have the same periods. The simplifications result in an equal or higher force amplitude than the force model given in the ISO 10137 standard, but always on the safe side. In some situations the simplifications might lead to a force amplitude well above the force amplitude in the standard.

The force model in the ISO 10137 standard gives the structural engineer many interpretation options. One is regarding the static weight of pedestrians which is not given and is not suggested in the standard. Also in the third design situation, where significantly more than fifteen pedestrians are walking on the bridge, it is up to the structural engineer to choose an appropriate number. For the case of Älvsbackabron the static weight of one pedestrian is assumed as 80 kg and fifty pedestrians are chosen for the third design situation.

The magnitude of the force calculated for Älvsbackabron according to BRO 2004 is recalculated to represent an equivalent amount of pedestrians on the bridge according to ISO 10137. The equivalent number of pedestrians is twenty and it is considered a possible loading situation on daily basis on Älvsbackabron, perhaps more relevant than a group of fifty pedestrians. If an amount of twenty pedestrians are chosen for the third design situation in ISO 10137 both the vertical and lateral acceleration limits given in Eurocode 0 are fulfilled. Thus, the amount pedestrians are decisive and it is up to the designer to choose a relevant amount of pedestrians.

Despite the fact that the magnitude of the force in ISO 10137 with fifty pedestrians is higher than the magnitude of the force in BRO 2004 the maximum vertical accelerations in the Master's thesis model according to BRO 2004 is higher than the vertical accelerations in the Master's thesis model according to Eurocode. The reason for this is the different damping factors used in the two Master's thesis models and the results show the importance of the damping factor.

Another interesting result is the different damping factors calculated from the jumping tests and the heel impact tests. From the jumping tests the damping factor is calculated to 0.6%, which is the same value as given in BRO 2004 for timber structures. However, from the heel impact test the damping factor is calculated to 1.2% which is twice as high. According to Eurocode 5, a damping factor of 1.5% can be used for structures with mechanical joints. This value is closer to the measured value from the heel impact test. A possible explanation to the different damping factors could be the cables on the bridge and the amount of energy inserted in the system by the different tests. The jumps in the jumping tests lasted for 30 seconds and during the jumping the cables started to sway. The sway in the cables continued over a minute after the jumping had stopped. This was probably what caused a lower damping factor. The cables absorbed the energy and kept the bridge moving by both increasing and decreasing the accelerations of the bridge deck. The forces in the cables could either be swaying in phase with the bridge accelerations and thus increase the accelerations, but they could also be counteracting and sway out of phase. The energy is kept in the system for a longer time due to the cables why a lower damping factor is calculated. In the heel impact test the force is only applied once as an impact and does not cause any significant sway in the cables why a higher damping is calculated. The higher value of the damping factor, closer to the Eurocode 5 value, should probably have been used in the design of Älvsbackabron since the effects from the cables are not included in linear design. This research could only indicate this relationship between the damping factor and the cables, why further studies and investigations regarding the effect are needed.

The maximum measured lateral acceleration is 0.14 m/s^2 which fulfils all the design requirements. This value is measured during the jumping tests where the jumping is done sideways. One difficulty when designing the tests was to excite the lateral modes of the bridge deck. The controlled walking test and the sideways jumping tests both aimed to excite the lateral modes, but it was hard to control the direction of the force and to know the exact magnitude of the component in lateral direction.

There are some possible error sources in the measurements at Älvsbackabron and the force modelling in the Master's thesis models. One is the impact from the wind which might have influence on the measurements. During the two test occasions the wind speed never exceeded five m/s^2 why its influence is neglected. A second source of error is the difficulty in jumping with the same frequency. This proved to be even harder when the bridge accelerations became too high. This resulted in that the force from the jumping group is not as perfectly harmonic as the force is simulated in the Master's thesis model according to Eurocode. However, the modelled results show reasonable correlation why the results are considered reliable. A third assumption that influences the result is the fact that the bridge is modelled as linear, but in reality it is not due to the influence of the cables, as an example.

The first two natural frequencies of the bridge are determined by FFT curves and are found to be between the theoretical values from the Master's thesis models according to BRO 2004 and Eurocode. The main differences between the codes that affect the natural frequencies are the densities of the timber and the glulam. Since the measured natural frequencies are in the range between the two theoretical frequencies calculated in the Master's thesis models the real density of the timber materials in the bridge are assumed to be between the theoretical densities as well. Some of the divergences concerning the natural frequencies may also depend on the fact that the modelled bridge does not have exactly the same properties as the built bridge. In Section 2.3 the uncertainties with FE modelling is described where also FE tuning of the models are suggested to increase the reliability of the model.

12 Conclusions

In this final chapter the conclusions of the Master's thesis are presented. As described in the first chapter, the report is divided in two parts, a literature study and a case study of Älvsbackabron. The purpose of the literature study is to find similarities and differences between the dynamic force models, regulations and comfort criteria presented in BRO 2004, Eurocode and the international standard ISO 10137.

One purpose of the case study of Älvsbackabron is to compare the resulting accelerations from the force models presented in BRO 2004 and Eurocode. Furthermore, accelerations are measured and from the results the damping factor of the bridge is calculated.

Finally, the general aim with the Master's thesis is to study how the transition from BRO 2004 to Eurocode affects the dynamic design of timber footbridges.

12.1 Regulations and force models

The vertical accelerations limits given in BRO 2004 and Eurocode 0 are fixed values with the same magnitude, namely 0.7 m/s^2. In the Danish standard Belastnings- og beregningsregler for vej- og stibroer the vertical acceleration limit is presented as a frequency dependent value. For frequencies below 4 Hz the limit is stricter than the limit in BRO 2004 and Eurocode 0.

The vertical limit in the ISO 10137 standard shows the opposite relation than the limit in the Danish standard with a higher limit value for frequencies below 2 Hz and a fixed limit of 0.4 m/s^2 for frequencies above 4 Hz. For the most common walking frequency, 2 Hz, the limits from BRO 2004, Eurocode 0 and ISO 10137 coincide. The Danish standard assigns a stricter limit for the same frequency.

No lateral acceleration limit is given in BRO 2004. In Eurocode 0 two fixed limits are given, one lower for normal use with a limit value of 0.2 m/s^2 and a higher limit of 0.4 m/s^2 for exceptional crowd conditions. The lateral acceleration limit in ISO 10137 has a fixed value of about 0.2 m/s^2 for frequencies below 2 Hz and is then frequency dependent with increasing limit values. In the Danish standard no specific lateral limit is given, but instead the same limit as for vertical accelerations are used. This results in a higher lateral acceleration limit than the other studied codes and standard, with an exception to the higher limit in Eurocode 0.

In Eurocode no force model for pedestrian force is presented other than for a design situation with a simply supported bridge deck, which is not relevant for the comparison. The vertical force models in BRO 2004 and the Danish standard are both harmonic forces with different force amplitudes. The force model in the ISO 10137 standard takes higher harmonics and the static weight of a pedestrian into account and therefore it is not a harmonic force. The result from the ISO 10137 model depends on the amount of pedestrians why it is of importance that relevant design situations for the bridge are assigned by the client or structural engineer.

It is only the ISO 10137 standard that presents a lateral force model. This model is the same as the vertical model, but with different coefficients.

Effects of the transition from BRO 2004 to the Eurocodes on the design of footbridges are that the lateral vibrations must be considered in design, the density for timber materials will be lower and the damping factor will be higher. The vertical acceleration limit in Eurocode 0 is the same as in BRO 2004, but a difference is that the ISO 10137 standard can also be used in design. Both the lateral and vertical limits are frequency dependent in the ISO 10137 standard. Since there is no general design method for dynamic design of cable-stayed timber footbridges in the Eurocodes, further development is needed.

12.2 Case study of Älvsbackabron

In the force model presented in ISO 10137 it is up to the client or structural engineer to assign an appropriate number of pedestrians to the design situation where a stream of pedestrians are crossing the bridge. For the case study, the force model in the ISO 10137 standard is simplified as a harmonic force and a stream of fifty pedestrians is chosen which results in a force amplitude of 1700N. The force amplitude from the force model given in BRO 2004 is 1082N. Though, the resulting vertical accelerations from the simplified ISO 10137 force model applied on the Master's thesis according to Eurocode are lower than the resulting accelerations from the BRO 2004 force applied on the Master's thesis model according to BRO 2004. This is due to the fact that a higher damping factor can be used according to Eurocode 5.

The resulting vertical accelerations that are calculated in both the Master's thesis models exceed the vertical acceleration limit according to BRO 2004 and Eurocode 0. On the other hand, from the measurements at Älvsbackabron the vertical acceleration limits in both BRO 2004 and Eurocode 0 are fulfilled by the measured accelerations from the continuous measurements and all tests except from the jumping test. However, the jumping tests are considered to be extreme loading situations and not a relevant design situation. In BRO 2004 no limit for the lateral acceleration is given, but none of the lateral limits given in Eurocode 0 are exceed by any of the measured lateral accelerations from the tests.

From the measured vertical accelerations at Älvsbackabron two damping factors are estimated based on the jumping tests and the heel impact tests. The jumping tests result in a damping factor of 0.6% and the heel impact tests in a damping factor of 1.2%. A possible explanation to the different damping factors is the amount of energy inserted in the bridge from the two tests. The jumping tests introduce a higher amount of energy to the bridge than the heel impact test. The high amount of energy from the jumping tests is absorbed by the cables which start to sway and the energy is kept in the bridge during a longer period of time which explains the lower damping factor. The results from the Master's thesis models of Älvsbackabron showed that the damping factor has the greatest influence on the accelerations. A damping factor of 1.2% is an appropriate value for further analyses of Älvsbackabron.

Effects of the transition from BRO 2004 to Eurocode on the design of footbridges are that the lateral vibrations must be considered in design, the density for timber materials will be lower and the damping factor will be higher. Since no force models for the pedestrian induced forces are given in Eurocode other than for the design case of a simply supported footbridge, the structural engineer is referred to the ISO 10137 standard. The force model presented in the ISO 10137 standard cannot be analysed in a frequency domain why simplifications are considered to be necessary.

13 References

Anonymous (2001): *Part 14 BD 37/01 Loads for Highway Bridges*, The Highways Agency, London.

Anonymous (2002a): *Belastnings- og beregningsregler for vej- og stibroer*, Vejdirektoratet.

Anonymous (2002b): *SS-EN 1990 Eurocode 0 – Basis of structural design*, 1st ed., Swedish Standards Institute, Stockholm.

Anonymous (2003): *SS-EN 1991-2 Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges*, 1st ed., Swedish Standards Institute, Stockholm.

Anonymous (2004a): *BD 29/04 Design Criteria for Footbridges*, The Highways Agency, London.

Anonymous (2004b): *SS-EN 1995-1-1:2004 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings*, 1st ed., Swedish Standards Institute, Stockholm.

Anonymous (2004c): *SS-EN 1995-2:2004 - Eurocode 5: Design of timber structures - Part2: Bridges*, 1st ed., Swedish Standards Institute, Stockholm.

Anonymous (2004d): *VV Publ 2004:56 BRO 2004 del 1 - Allmänna förutsättningar*, Vägverket, Borlänge.

Anonymous (2005): *SS-EN 1990/A1:2005 Eurocode 0: Basis of structural design*, 1st ed., Swedish Standards Institute, Stockholm.

Anonymous (2007): *ISO 10137:2007 Bases for design of structures – Serviceability of buildings and walkways against vibration,* 1st ed., International Organization for Standardization, Geneva.

Anonymous (2010): *Design of bridges to Eurocode*. [Online] Available at: http://eurocodes.jrc.ec.europa.eu/showevent.php?id=474 [2012-05-04]

Anonymous (2011): *TRVFS 2011:12 Trafikverketsföreskrifter om ändring i Vägverkets föreskrifter (VVFS 2004:43) om tillämpningen av europeiska beräkningsstandarder,* Trafikverket, Borlänge.

Anonymous (2012): *Technical Cooperation between ISO and CEN*. [Online] Available at: http://eurocodes.jrc.ec.europa.eu/showpage.php?id=145 [2010-05-03]

Bachmann, H., Ammann, W. (1987): *Vibrations in structures*, IABSE-AIPC-IVBH, Zurich.

Byggindustrin (2010): *Träbro byggs i Skellefteå*. [Online] Available at: http://www.byggindustrin.com/byggprojekt/byggandet-av-trabron-i-skelleftea-drar-i__7979 [2012-01-10]

Craig Jr, R.R., Kurdila, A.J. (2006): *Fundamentals of structural dynamics,* 2nd ed., John Wiley & Sons, Inc, Hoboken.

Dallard, P., Fitzpatrick, T., Flint, A., Low, A., Ridsdill Smith, R., Willford, M., Roche, M., (2001a): London Millenium Bridge: Pedestrian-induced lateral vibration. *Journal of Bridge Engineering*, Vol. 6, No. 6, 2001, pp.412-17.

Dallard, P., Fitzpatrick, T., Flint, A., Low, A., Ridsdill Smith, R., Willford, M., Roche, M., (2001b): The London Millenium Footbridge. *The Structural Engineer*, Vol. 79, No. 22, 2001, pp. 17-33.

Degerfeldt, K. (2009): *Fördjupad projektbeskrivning "Smart Träbro i Smart Stad"*, Skellefteå kommun, Skellefteå.

Dhyr, J. (2012): Hopp för bron [Online] *Norran*, 8:th of May. Available at: http://norran.se/2012/05/skelleftea/hopp-for-bron/. [2012-05-25]

Eriksson, A. (2012): Hopp för bron [Online] *Norran*, 8:th of May. Available at: http://norran.se/2012/05/skelleftea/hopp-for-bron/. [2012-05-25]

Heinemeyer, C., Butz, C., Keil, A., Mark, S., Goldack, A., Trometer, S., Lukić, M., Chabrolin, B., Lemaire, A., Martin, P., Cunha, Á., Caetano, E., (2009). *Design of Lightweight Footbridges for Human Induced Vibrations. Background document in support to the implementation, harmonization and further development of the Eurocodes*, JRC European Commission, Luxembourg.

Ingólfsson, E.T., Georgakis, C.T. (2011): A stochastic load model for pedestrian-induced lateral forces on footbridges. *Engineering Structures*, No. 33, pp.3454-3470.

Jacobsson, P. (2009): *Avsiktsförklaring om medverkan i projektet "Smart Träbro i Smart Stad"*, Martinsons Träbroar AB, Skellefteå.

Jürisoo, M., Ohlsson, S., Edlund, B. (1980): *Vibrationsmätningar på fyra gångbroar av stål*, Chalmers tekniska högskola, Göteborg.

Maguire, J.R., Wyatt, T. (2002): *Dynamics: an introduction for civil and structural engineers*, 2nd ed., Thomas Telford Publishing, London.

Martinsons (2009): Älvsbackabron. *Nytt från Martinsons*, No. 1/09, 2009, pp. 1-9.

Martinsons (2012): *Om företaget*. [Online] Available at: http://martinsons.se/om-foretaget [2012-01-12]

McConnell, K.G. (2001): Modal testing. *Philosophical transactions of the royal society*, No. 359, 2001, pp.11-28.

Melchor Blanco, C., Bouillard, P., Bodarwé, E., Ney, L. (2005): Structural dynamic design of a footbridge. *SAMTECH Users Conference, 2005*, pp. 1-19.

Mynewsdesk (2010): *Byggstart för träbro i världsklass - Skellefteå kommun*. [Online] Available at: http://www.mynewsdesk.com/se/view/pressrelease/byggstart-foer-traebro-i-vaerldsklass-450274 [2012-01-12]

Nakamura, S., Kawasaki, T. (2006): Lateral vibration of footbridges by synchronous walking. *Journal of Constructional Steel Research*, No. 62, 2006, pp.1148-1160.

Nakamura, S., Kawasaki, T. (2009): A method for predicting the lateral girder response of footbridges induced by pedestrians. *Journal of Constructional Steel Research*, No. 65, 2009, pp.1705-1711.

Nationalencyklopedin (2012): *Skellefteå*. [Online] Available at: http://www.ne.se.proxy.lib.chalmers.se/lang/skellefteå/307453 [2012-01-12]

Occhiuzzi, A., Spizzuoco, M., Ricciardelli, F. (2008): Loading models and response control of footbridges excited by running pedestrian. *Structural control health monitoring*, No. 15, 2008, pp.349-368.

Ohlsson, S. (1995): *Modsyntes baserad på dynamisk provning,* Chalmers University of Technology, Göteborg.

Pousette, A. (2001): *Cable stayed timber bridges*, Trätek, Stockholm.

Puranen, M. (2011): *Övre Norrland*. [Online] Available at: http://www.tillvaxtverket.se/huvudmeny/euprogram/ovrenorrland.4.21099e4211fdba8c87b800017079.html [2012-01-13]

Stoyanoff, S., Hunter, M. (2003): Footbridges: Pedestrian induced vibrations. *Tech Notes,* No. 15, 2003, pp. 1-2.

Trafikverket (2012): *ATB - Bro 2004*. [Online] Available at: http://www.trafikverket.se/Foretag/Bygga-och-underhalla/Vag/Tekniska-dokument/Bro-och-tunnel/Allmanna-tekniska-beskrivningar/ATB---Bro-2004/ [2012-04-26].

Trästad (2012): *Skellefteå-region nord*. [Online] Available at: http://www.trastad2012.se/web/Skelleftea.aspx [2012-01-12].

Wheeler, J.E. (1982): Prediction and control of pedestrian induced vibration in footbridges. *Journal of the Structural Division*, No. 108, 1982, pp.2045-2065.

Živanović, S., Pavić, A., Reynolds, P. (2005): Vibration serviceability of footbridges under human-induced excitation: a literature review. *Journal of Sound and Vibration*, Vol. 279, No. 1-2, 2005, pp.1-74.

Živanović, S., Pavic, A., Reynolds, P. (2006). Modal testing and FE model tuning of a lively footbridge structure. *Engineering Structures,* No. 28, 2006, pp.857-868.

Appendix A

This appendix describes how to produce the Master's thesis model in Brigade/Plus 3.1-5.

Go to Module: Part

Press Part -> Create. Mark 3D, Deformable, Shell and Extrusion.

Press Add -> Spline. The spline have the coordinates (0,0), (65,1.057) and (130,0). Exit the sketcher.

Choose the Depth to 4.185.

Press Shape -> Wire -> Spline. The spline have the coordinates (0,-0.3575,-0.3225), (65,0.6995,-0.3225) and (130,-0.3575,-0.3225).

Press Shape -> Wire -> Spline. The spline have the coordinates (0,-0.3575,4.5075), (65,0.6995,4.5075) and (130,-0.3575,4.5075).

Press Shape -> Wire -> Point to Point. Add the wires with the coordinates in Table A.1.

Table A.1 Coordinates for the glulam cross beams and the pylons.

Point 1	Point 2
0.0,-0.357500,4.5075	0.0,-0.357500,-0.3225
2.5,-0.296549,4.5075	2.5,-0.296549,-0.3225
5.0,-0.235779,4.5075	5.0,-0.235779,-0.3225
7.5,-0.175370,4.5075	7.5,-0.175370,-0.3225
10.0,-0.115501,4.5075	10.0,-0.115501,-0.3225
12.5,-0.056355,4.5075	12.5,-0.056355,-0.3225
15.0,0.001890,4.5075	15.0,0.001890,-0.3225
17.5,0.059052,4.5075	17.5,0.059052,-0.3225
20.0,0.114951,4.5075	20.0,0.114951,-0.3225
22.5,0.169406,4.5075	22.5,0.169406,-0.3225
25.0,0.222238,4.5075	25.0,0.222238,-0.3225
27.5,0.273266,4.5075	27.5,0.273266,-0.3225
30.0,0.322309,4.5075	30.0,0.322309,-0.3225
32.5,0.369187,4.5075	32.5,0.369187,-0.3225

35.0,0.413720,4.5075	35.0,0.413720,-0.3225
37.5,0.455727,4.5075	37.5,0.455727,-0.3225
40.0,0.495028,4.5075	40.0,0.495028,-0.3225
42.5,0.531442,4.5075	42.5,0.531442,-0.3225
45.0,0.564789,4.5075	45.0,0.564789,-0.3225
47.5,0.594889,4.5075	47.5,0.594889,-0.3225
50.0,0.621560,4.5075	50.0,0.621560,-0.3225
52.5,0.644623,4.5075	52.5,0.644623,-0.3225
55.0,0.663898,4.5075	55.0,0.663898,-0.3225
57.5,0.679203,4.5075	57.5,0.679203,-0.3225
60.0,0.690359,4.5075	60.0,0.690359,-0.3225
62.5,0.697185,4.5075	62.5,0.697185,-0.3225
65.0,0.699500,4.5075	65.0,0.699500,-0.3225
67.5,0.697185,4.5075	67.5,0.697185,-0.3225
70.0,0.690359,4.5075	70.0,0.690359,-0.3225
72.5,0.679203,4.5075	72.5,0.679203,-0.3225
75.0,0.663898,4.5075	75.0,0.663898,-0.3225
77.5,0.644623,4.5075	77.5,0.644623,-0.3225
80.0,0.621560,4.5075	80.0,0.621560,-0.3225
82.5,0.594889,4.5075	82.5,0.594889,-0.3225
85.0,0.564789,4.5075	85.0,0.564789,-0.3225
87.5,0.531442,4.5075	87.5,0.531442,-0.3225
90.0,0.495028,4.5075	90.0,0.495028,-0.3225
92.5,0.455727,4.5075	92.5,0.455727,-0.3225
95.0,0.413720,4.5075	95.0,0.413720,-0.3225
97.5,0.369187,4.5075	97.5,0.369187,-0.3225

100.0,0.322309,4.5075	100.0,0.322309,-0.3225
102.5,0.273266,4.5075	102.5,0.273266,-0.3225
105.0,0.222238,4.5075	105.0,0.222238,-0.3225
107.5,0.169406,4.5075	107.5,0.169406,-0.3225
110.0,0.114951,4.5075	110.0,0.114951,-0.3225
112.5,0.059052,4.5075	112.5,0.059052,-0.3225
115.0,0.001890,4.5075	115.0,0.001890,-0.3225
117.5,-0.056355,4.5075	117.5,-0.056355,-0.3225
120.0,-0.115501,4.5075	120.0,-0.115501,-0.3225
122.5,-0.175370,4.5075	122.5,-0.175370,-0.3225
125.0,-0.235779,4.5075	125.0,-0.235779,-0.3225
127.5,-0.296549,4.5075	127.5,-0.296549,-0.3225
130.0,-0.357500,4.5075	130.0,-0.357500,-0.3225
130.0,4.731,5.9925	130.0,4.731,-1.8075
130.0,10.231,5.9925	130.0,10.231,-1.8075
130.0,15.731,5.9925	130.0,15.731,-1.8075
130.0,21.231,5.9925	130.0,21.231,-1.8075
0.0,21.231,5.9925	0.0,21.231,-1.8075
0.0,10.231,5.9925	0.0,10.231,-1.8075
0.0,15.731,5.9925	0.0,15.731,-1.8075
0.0,4.731,5.9925	0.0,4.731,-1.8075
0,-1.026,-1.8075	0,23.055,-1.8075
0,-1.026,5.9925	0,23.055,5.9925
130,-1.026,-1.8075	130,23.055,-1.8075
130,-1.026,5.9925	130,23.055,5.9925

Press Shape -> Wire -> Point to Point . Add the wires with the coordinates in Table A.2.

Table A.2 Coordinates for the steel cross beams

First point	Second point	Third point	Fourth point
16.25,-0.846883,5.3025	16.25,-0.846883,4.5075	16.25,-0.846883,0.3225	16.25,-0.846883,-1.1175
32.50,-0.508313,5.3025	32.50,-0.508313,4.5075	32.50,-0.508313,0.3225	32.50,-0.508313,-1.1175
48.75,-0.268836,5.3025	48.75,-0.268836,4.5075	48.75,-0.268836,0.3225	48.75,-0.268836,-1.1175
65.00,-0.178000,5.3025	65.00,-0.178000,4.5075	65.00,-0.178000,0.3225	65.00,-0.178000,-1.1175
81.25,-0.268836,5.3025	81.25,-0.268836,4.5075	81.25,-0.268836,0.3225	81.25,-0.268836,-1.1175
97.50,-0.508313,5.3025	97.50,-0.508313,4.5075	97.50,-0.508313,0.3225	97.50,-0.508313,-1.1175
113.75,-0.846883,5.3025	113.75,-0.846883,4.5075	113.75,-0.846883,0.3225	113.75,-0.846883,-1.1175

Press Shape -> Wire -> Point to Point. Add the wires with the coordinates in Table A.3.

Press Shape -> Wire -> Spline. Add the wires with the coordinates in Table A.4.

Table A.3 Coordinates for the Point-to-point wires in the cross bracings

Point 1	Point 2
0,21.231,5.9925	0,15.731,-1.8075
0,15.731,5.9925	0,10.231,-1.8075
0,10.231,5.9925	0,4.731,-1.8075
130,21.231,5.9925	130,15.731,-1.8075
130,15.731,5.9925	130,10.231,-1.8075
130,10.231,5.9925	130,4.731,-1.8075
2.5,-0.296549,4.5075	7.5,-0.175370,-0.3225
7.5,-0.175370,4.5075	12.5,-0.056355,-0.3225
12.5,-0.056355,4.5075	17.5,0.059052,-0.3225
17.5,0.059052,4.5075	22.5,0.169406,-0.3225
22.5,0.169406,4.5075	27.5,0.273266,-0.3225
27.5,0.273266,4.5075	32.5,0.369187,-0.3225
32.5,0.369187,4.5075	37.5,0.455727,-0.3225
37.5,0.455727,4.5075	42.5,0.531442,-0.3225
42.5,0.531442,4.5075	47.5,0.594889,-0.3225
47.5,0.594889,4.5075	52.5,0.644623,-0.3225
52.5,0.644623,4.5075	57.5,0.679203,-0.3225
57.5,0.679203,4.5075	62.5,0.697185,-0.3225
67.5,0.697185,4.5075	72.5,0.679203,-0.3225
72.5,0.679203,4.5075	77.5,0.644623,-0.3225
77.5,0.644623,4.5075	82.5,0.594889,-0.3225
82.5,0.594889,4.5075	87.5,0.531442,-0.3225
87.5,0.531442,4.5075	92.5,0.455727,-0.3225
92.5,0.455727,4.5075	97.5,0.369187,-0.3225

97.5,0.369187,4.5075	102.5,0.273266,-0.3225
102.5,0.273266,4.5075	107.5,0.169406,-0.3225
107.5,0.169406,4.5075	112.5,0.059052,-0.3225
112.5,0.059052,4.5075	117.5,-0.056355,-0.3225
117.5,-0.056355,4.5075	122.5,-0.175370,-0.3225
122.5,-0.175370,4.5075	127.5,-0.296549,-0.3225

Table A.4 Coordinates for the spline wires in the cross bracings

Start point	Next point	Next point
0,21.231,-1.8075	0.000001,18.481,2.0925	0,15.731,5.9925
0,15.731,-1.8075	0.000001,12.981,2.0925	0,10.231,5.9925
0,10.231,-1.8075	0.000001,7.481,2.0925	0,4.731,5.9925
130,21.231,-1.8075	130.000001,18.481,2.092 5	130,15.731,5.9925
130,15.731,-1.8075	130.000001,12.981,2.092 5	130,10.231,5.9925
130,10.231,-1.8075	130.000001,7.481,2.0925	130,4.731,5.9925
2.5,-0.296549,-0.3225	5.00, -0.235950,2.0925	7.5,-0.175370,4.5075
7.5,-0.175370,-0.3225	10.0, -0.115852,2.0925	12.5,-0.056355,4.5075
12.5,-0.056355,-0.3225	15.0, 0.001338,2.0925	17.5,0.059052,4.5075
17.5,0.059052,-0.3225	20.0, 0.114219,2.0925	22.5,0.169406,4.5075
22.5,0.169406,-0.3225	25.0, 0.221326,2.0925	27.5,0.273266,4.5075
27.5,0.273266,-0.3225	30.0, 0.321217,2.0925	32.5,0.369187,4.5075
32.5,0.369187,-0.3225	35.0, 0.412447,2.0925	37.5,0.455727,4.5075
37.5,0.455727,-0.3225	40.0, 0.493574,2.0925	42.5,0.531442,4.5075
42.5,0.531442,-0.3225	45.0, 0.563155,2.0925	47.5,0.594889,4.5075
47.5,0.594889,-0.3225	50.0, 0.619746,2.0925	52.5,0.644623,4.5075
52.5,0.644623,-0.3225	55.0, 0.661903,2.0925	57.5,0.679203,4.5075

57.5,0.679203,-0.3225	60.0, 0.688184,2.0925	62.5,0.697185,4.5075
67.5,0.697185,-0.3225	70.0, 0.688184,2.0925	72.5,0.679203,4.5075
72.5,0.679203,-0.3225	75.0, 0.661903,2.0925	77.5,0.644623,4.5075
77.5,0.644623,-0.3225	80.0, 0.619746,2.0925	82.5,0.594889,4.5075
82.5,0.594889,-0.3225	85.0, 0.563155,2.0925	87.5,0.531442,4.5075
87.5,0.531442,-0.3225	90.0, 0.493574,2.0925	92.5,0.455727,4.5075
92.5,0.455727,-0.3225	95.0, 0.412447,2.0925	97.5,0.369187,4.5075
97.5,0.369187,-0.3225	100.0, 0.321217,2.0925	102.5,0.273266,4.5075
102.5,0.273266,-0.3225	105.0, 0.221326,2.0925	107.5,0.169406,4.5075
107.5,0.169406,-0.3225	110.0, 0.114219,2.0925	112.5,0.059052,4.5075
112.5,0.059052,-0.3225	115.0, 0.001338,2.0925	117.5,-0.056355,4.5075
117.5,-0.056355,-0.3225	120.0, -0.115852,2.0925	122.5,-0.175370,4.5075
122.5,-0.175370,-0.3225	125.0, -0.235950,2.0925	127.5,-0.296549,4.5075

Go to Module: Property

Press Material -> Create. Press General -> Density and write 430 in Mass Density. Press Mechanical -> Elasticity -> Elastic and write 13E9 in Young’s Modulus and 0.4 in Poisson’s Ratio.

Press Material -> Create. Press General -> Density and write 7850 in Mass Density. Press Mechanical -> Elasticity -> Elastic and write 210E9 in Young’s Modulus and 0.3 in Poisson’s Ratio.

Press Material -> Create. Press General -> Density and write 7850 in Mass Density. Press Mechanical -> Elasticity -> Elastic and write 200E9 in Young’s Modulus and 0.3 in Poisson’s Ratio. Check No compression

Press Material -> Create. Press General -> Density and write 454 in Mass Density. Press Mechanical -> Elasticity -> Elastic and change Type to Lamina. Write 4846E6 in E1, 4846E3 in E2, 0 in Nu12, 323E3 in G12, 323E6 in G13 and 323E3 in G23.

Press Profile -> Create. Mark Rectangular and press continue. Write 0.215 in a and 0.495 in b.

Press Profile -> Create. Mark Box and press continue. Write 0.4 in a and b and 0.016 in t.

Press Profile -> Create. Mark Trapezoidal and press continue. Write 0.645 in a and c, 1.125 in b and 0.4475 in d.

Press Profile -> Create. Mark Rectangular and press continue. Write 0.86 in a and 0.9 in b.

Press Section –> Create. Mark Shell and Homogeneous and press continue. Write 0.22 in Shell thickness and choose Timber as Material.

Press Section –> Create. Mark Beam and Beam and press continue. Choose Pylons as profile and glulam as material. Repeat for main beams and cross beams.

Press Section –> Create. Mark Beam and Truss. Write cross-sectional area as in Table A.5.

Table A.5 Cross-sectional areas for cables and bracings

Part	Cross-sectional area
Cable M48	0.00180956
Cable M64	0.00321699
Cable M80	0.0100530
Bracing M24	0.000226195
Bracing M30	0.000353429
Bracing M36	0.00050894
Bracing M42	0.00069272
Bracing M48	0.00090478
Bracing M56	0.00123151

Press Section -> Assignment Manager. Press create, choose the Pylons in the viewport and choose Pylon as profile. Repeat for deck, main beams, cross beams, cables and bracings.

Press Assign -> Beam Section Orientation. Mark all cables, bracings and the pylons and press done. Accept the predefined (0.0,0.0,-1.0).

Press Assign -> Beam Section Orientation. Mark all the cross beams and press done. Write “1.0,0.0,0.0” in the textbox.

Press Assign -> Beam Section Orientation. Mark the two main beams and press done. Write “0.0,0.0,1.0” in the textbox.

Press Special -> Springs/Dashpots -> Create. Mark Connect points to ground (standard) and choose the end of both the main beams with x-coordinate 0. Select Degree of freedom 2 and write 625E6 in Spring stiffness.

Press Special -> Springs/Dashpots -> Create. Mark Connect points to ground (standard) and choose the other end of both the main beams. Select Degree of freedom 2 and write 15625E4 in Spring stiffness.

Press Special -> Springs/Dashpots -> Create. Mark Connect points to ground (standard) and choose the bottom of the pylons with x-coordinate 0. Select Degree of freedom 1 and write 82E6 in Spring stiffness.

Press Special -> Springs/Dashpots -> Create. Mark Connect points to ground (standard) and choose the bottom of the pylons with x-coordinate 0. Select Degree of freedom 2 and write 625E6 in Spring stiffness.

Press Special -> Springs/Dashpots -> Create. Mark Connect points to ground (standard) and choose the bottom of the pylons with x-coordinate 130. Select Degree of freedom 1 and write 335E5 in Spring stiffness.

Press Special -> Springs/Dashpots -> Create. Mark Connect points to ground (standard) and choose the bottom of the pylons with x-coordinate 130. Select Degree of freedom 2 and write 15625E4 in Spring stiffness.

Press Special -> Inertia -> Create. Mark Nonstructural mass and select both main beams. Choose Mass per Length and write 70 in magnitude.

Press Special -> Inertia -> Create. Mark Nonstructural mass and select all the pylons. Choose Mass per Length and write 83.4 in magnitude.

Press Special -> Inertia -> Create. Mark Nonstructural mass and select the cross beams in the pylons. Choose Mass per Length and write 16.7 in magnitude.

Go to Module: Assembly

Press Instance -> Create. Choose the part and mark Independent (mesh on instance) as Instance Type.

Go to Module: Step

Press Step -> Create. Mark Initial, choose Linear Perturbation and mark Frequency and press continue. Choose Value in Number of eigenvalues requested and write 20.

Go to Module: Interaction

Press Constraint -> Create. Mark Tie and press continue, press Surface as master type and choose the bridge deck. Choose the colour that represents the downside of the deck, choose Node Region as slave and mark all the glulam cross beams in the deck. Unmark Adjust slave surface initial position and Tie rotational DOFs if applicable and Specify distance to 1. Choose Node to surface as Discretization method and press OK.

Press Constraint -> Create. Mark Tie and press continue, choose Node surface as master and pick one of the main beams. Choose Node surface as slave and pick the points on the steel cross beams right under the main beams. Unmark Adjust slave surface initial position and Specify distance to 1. Choose Node to surface as Discretization method and press OK. Repeat the same procedure for the other main beam.

Go to Module: Load

Press BC -> Create, Mark Mechanical and Displacement/Rotation and press continue. Select the short end of the deck where the x-coordinate is 0 and restrain U1 and U3.

Press BC -> Create, Mark Mechanical and Displacement/Rotation and press continue. Select the other short end of the deck restrain U3.

Press BC -> Create, Mark Mechanical and Displacement/Rotation and press continue. Select the bottom of all pylons and restrain U3.

Go to Module: Mesh

Press Seed -> Instance and choose 0.5 as approximate size.

Press Seed -> Edge by number and mark all cables and bracing elements. Choose 1 element.

Press Mesh -> Element type and mark all cables and bracing elements. Choose truss under family.

Press Mesh -> Instance and press yes.

Go to Module: Job

Press Job -> Create and create the job.

Press Job -> Manager and submit the job.

Appendix B

NORRAN

http://norran.se

UTSKRIFTEN SPONSRAS AV: SHARP VÄSTERBOTTEN 0910-71 14 20

Hopp för bron

av:

Jessica Dhyr jessica.dhyr@norran.se

Publicerad: Tisdag 08 maj   Senaste nytt,Skellefteå

Ett, två, tre. Under tisdagen kunde förbipasserande se ett gäng som hoppade på Älvsbackabron. Allt för ett forskningsprojekt. Foto: Annika Eriksson

Nordens längsta snedstagsbro i trä får under tisdagen genomgå flera olika tester och mätningar. Det pågår nämligen ett forskningsprojekt som drivs av Martinssons träbroar, SP Trätek, LTU, Cowi och Chalmers.

Hanna Jansson studerar på Chalmers och bromätningarna är hennes examensarbete.

– Vi håller på att utvärdera brons dynamiska respons, det vill säga hur gångtrafiken påverkar svängningarna på bron.

För att göra det hoppade ett tiotal upp och ner fler gånger i samma takt samtidigt som mätutrustning registrerade hur bron reagerade på hoppen.

Peter Jacobsson, Martinssons träbroar, tycker att forskningsprojektet är spännande. För hans företag handlar det främst om att lära sig så mycket som möjligt inför framtida träbroar. Trots att det inte kommit fram så mycket mätningsresultat än, så han på det stora hela nöjd med bron.

– Om man ser till det verkliga så har vi inte fått in klagomål från folk som faktiskt går på bron, säger Peter Jacobsson.

URL: **http://norran.se/2012/05/skelleftea/hopp-for-bron/**

Appendix C

— Calibration Certificate —
Per ISA-RP37.2

Model No. 393B12
Serial No. 4364
PO No. Customer
Calibration traceable to NIST thru Project No. 822/255630

ICP® ACCELEROMETER
with built-in electronics

Calibration procedure is in compliance with ISO 10012-1, and former MIL-STD-45662A and traceable to NIST.

CALIBRATION DATA

Voltage Sensitivity	9711	mV/g
Transverse Sensitivity	4.7	%
Resonant Frequency	13.0	kHz
Output Bias Level	9.4	V
Time Constant	≥3.5	s

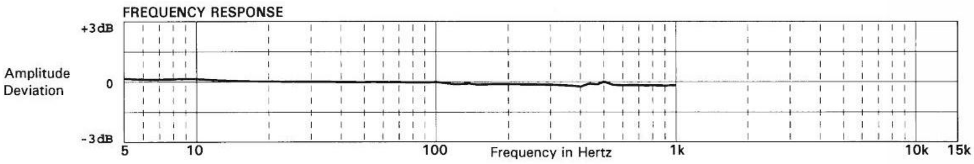
KEY SPECIFICATIONS

Range	0.5	±g
Resolution	0.000008	g
Temp. Range	-50/+180	°F

METRIC CONVERSIONS:
ms^{-2} = 0.102 g
°C = 5/9 x (°F - 32)

Reference Freq.

Frequency Hz	5	10	15	30	50	100	300	500	1000						
Amplitude Deviation %	2.0	1.7	0.5	0.2	-0.1	0.0	-1.6	0.1	-1.9						



PCB® Piezotronics, Inc. 3425 Walden Avenue Depew, NY 14043-2495 USA
716-684-0001

Calibrated by Andrew Shwec
Date 12-23-1996

— Calibration Certificate —
Per ISA-RP37.2

Model No. 393B12
Serial No. 4425
PO No. Customer
Calibration traceable to NIST thru Project No. 822/255630

ICP® ACCELEROMETER
with built-in electronics

Calibration procedure is in compliance with ISO 10012-1, and former MIL-STD-45662A and traceable to NIST.

CALIBRATION DATA

Voltage Sensitivity	9887	mV/g
Transverse Sensitivity	4.0	%
Resonant Frequency	12.0	kHz
Output Bias Level	10.7	V
Time Constant	≥3.5	s

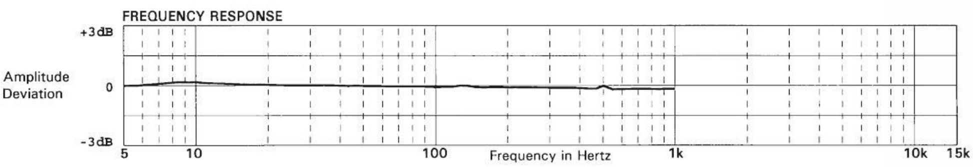
KEY SPECIFICATIONS

Range	0.5	±g
Resolution	0.000008	g
Temp. Range	-50/+180	°F

METRIC CONVERSIONS:
ms^{-2} = 0.102 g
°C = 5/9 x (°F - 32)

Reference Freq.

Frequency Hz	5	10	15	30	50	100	300	500	1000						
Amplitude Deviation %	0.0	2.1	0.8	0.2	-0.2	0.0	-1.2	-0.1	-1.8						



PCB® Piezotronics, Inc. 3425 Walden Avenue Depew, NY 14043-2495 USA
716-684-0001

Calibrated by Andrew Shwec
Date 01-14-1997

— Calibration Certificate —

Per ISA-RP37.2

Model No. 393B12

Serial No. 4434

PO No. ______ Customer ______

Calibration traceable to NIST thru Project No. 822/255630

ICP® ACCELEROMETER
with built-in electronics

Calibration procedure is in compliance with ISO 10012-1, and former MIL-STD-45662A and traceable to NIST.

CALIBRATION DATA

Voltage Sensitivity	9746	mV/g
Transverse Sensitivity	2.6	%
Resonant Frequency	12.0	kHz
Output Bias Level	10.9	V
Time Constant	≥3.5	s

KEY SPECIFICATIONS

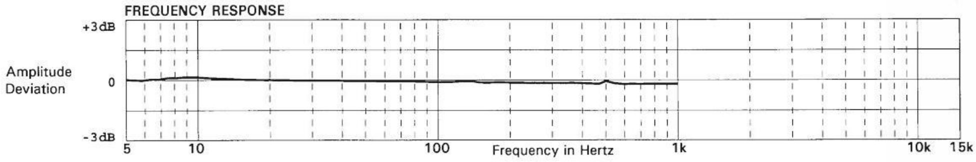
Range	0.5	±g
Resolution	0.000008	g
Temp. Range	-50/+180	°F

METRIC CONVERSIONS:
ms^{-2} = 0.102 g
°C = 5/9 x (°F - 32)

Reference Freq.

Frequency Hz	5	10	15	30	50	100	300	500	1000						
Amplitude Deviation %	0.4	1.7	0.5	-0.2	-0.5	0.0	-1.6	-0.6	-2.2						

FREQUENCY RESPONSE



PCB® Piezotronics, Inc. 3425 Walden Avenue Depew, NY 14043-2495 USA
716-684-0001

Calibrated by Andrew Shwec

Date 01-15-1997

— Calibration Certificate —

Per ISA-RP37.2

Model No. 393B12

Serial No. 4345

PO No. ______ Customer ______

Calibration traceable to NIST thru Project No. 822/255630

ICP® ACCELEROMETER
with built-in electronics

Calibration procedure is in compliance with ISO 10012-1, and former MIL-STD-45662A and traceable to NIST.

CALIBRATION DATA

Voltage Sensitivity	9696	mV/g
Transverse Sensitivity	6.0	%
Resonant Frequency	14.0	kHz
Output Bias Level	8.9	V
Time Constant	≥3.5	s

KEY SPECIFICATIONS

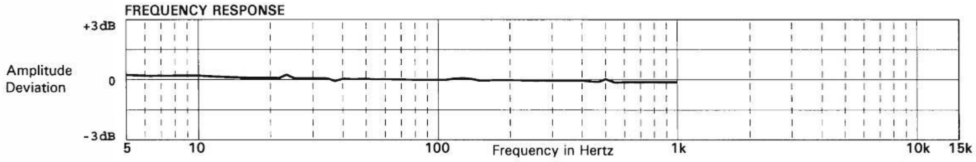
Range	0.5	±g
Resolution	0.000008	g
Temp. Range	-50/+180	°F

METRIC CONVERSIONS:
ms^{-2} = 0.102 g
°C = 5/9 x (°F - 32)

Reference Freq.

Frequency Hz	5	10	15	30	50	100	300	500	1000						
Amplitude Deviation %	2.7	2.3	1.2	0.8	0.6	0.0	-0.6	0.3	-1.4						

FREQUENCY RESPONSE



PCB® Piezotronics, Inc. 3425 Walden Avenue Depew, NY 14043-2495 USA
716-684-0001

Calibrated by Andrew Shwec

Date 12-19-1996

— Calibration Certificate —

Per ISA-RP37.2

Model No. V356A11/M034AD010AC

Serial No. 3646 (Y-AXIS)

PO No. ______ Customer ______

Calibration traceable to NIST thru Project No. 822/256889

ICP® ACCELEROMETER
with built-in electronics

Calibration procedure is in compliance with ISO 10012-1, and former MIL-STD-45662A and traceable to NIST.

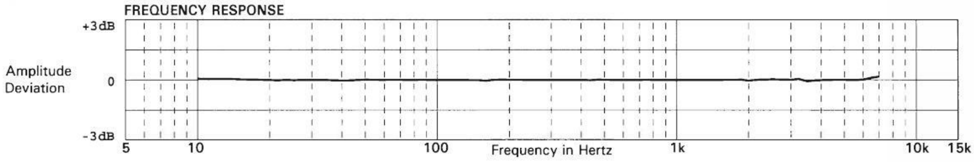
CALIBRATION DATA		
Voltage Sensitivity	9.91	mV/g
Transverse Sensitivity	4.9	%
Resonant Frequency	≥35	kHz
Output Bias Level	8.1	V
Time Constant	0.4	s

KEY SPECIFICATIONS		
Range	500	±g
Resolution	0.002	g
Temp. Range	-65/+250	°F

METRIC CONVERSIONS:
ms^{-2} = 0.102 g
°C = 5/9 x (°F - 32)

Reference Freq.

Frequency Hz		10	15	30	50	100	300	500	1000	3000	5000	7000			
Amplitude Deviation %		0.7	0.3	-0.0	0.1	0.0	0.0	0.0	-0.1	0.1	0.0	2.0			



PCB® Piezotronics, Inc. 3425 Walden Avenue Depew, NY 14043-2495 USA
716-684-0001

Calibrated by Martin Coleman
Date 03-11-1997

— Calibration Certificate —

Per ISA-RP37.2

Model No. V356A11/M034AD010AC

Serial No. 3646 (Z-AXIS)

PO No. ______ Customer ______

Calibration traceable to NIST thru Project No. 822/256889

ICP® ACCELEROMETER
with built-in electronics

Calibration procedure is in compliance with ISO 10012-1, and former MIL-STD-45662A and traceable to NIST.

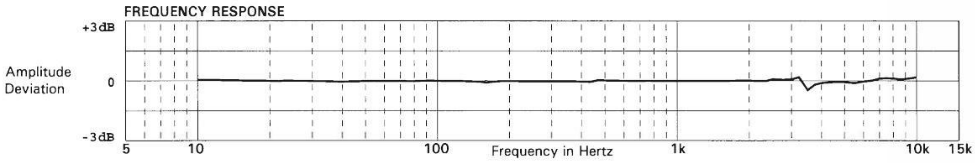
CALIBRATION DATA		
Voltage Sensitivity	10.53	mV/g
Transverse Sensitivity	3.8	%
Resonant Frequency	≥35	kHz
Output Bias Level	8.1	V
Time Constant	0.4	s

KEY SPECIFICATIONS		
Range	500	±g
Resolution	0.002	g
Temp. Range	-65/+250	°F

METRIC CONVERSIONS:
ms^{-2} = 0.102 g
°C = 5/9 x (°F - 32)

Reference Freq.

Frequency Hz		10	15	30	50	100	300	500	1000	3000	5000	7000	10000		
Amplitude Deviation %		0.6	0.3	-0.1	-0.0	0.0	-0.2	0.2	-0.1	0.8	-0.7	1.4	2.1		



PCB® Piezotronics, Inc. 3425 Walden Avenue Depew, NY 14043-2495 USA
716-684-0001

Calibrated by Martin Coleman
Date 03-11-1997

— Calibration Certificate —

Per ISA-RP37.2

Model No. V356A11/M034AD010AC

Serial No. 3647 (Y-AXIS)

PO No. ____________ Customer ____________

Calibration traceable to NIST thru Project No. 822/256889

ICP® ACCELEROMETER
with built-in electronics

Calibration procedure is in compliance with ISO 10012-1, and former MIL-STD-45662A and traceable to NIST.

CALIBRATION DATA

Voltage Sensitivity	10.49	mV/g
Transverse Sensitivity	3.4	%
Resonant Frequency	≥35	kHz
Output Bias Level	8.3	V
Time Constant	0.4	s

KEY SPECIFICATIONS

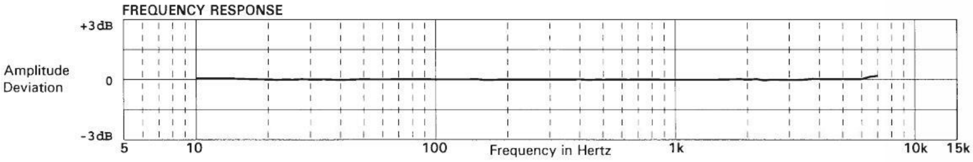
Range	500	±g
Resolution	0.002	g
Temp. Range	-65/+250	°F

METRIC CONVERSIONS:
ms^{-2} = 0.102 g
°C = 5/9 x (°F - 32)

Reference Freq.

Frequency Hz		10	15	30	50	100	300	500	1000	3000	5000	7000			
Amplitude Deviation %		0.5	0.4	-0.0	0.2	0.0	0.0	-0.1	-0.1	-0.4	0.3	2.2			

FREQUENCY RESPONSE



PCB® Piezotronics, Inc. 3425 Walden Avenue Depew, NY 14043-2495 USA
716-684-0001

Calibrated by Martin Coleman
Date 03-11-1997

— Calibration Certificate —

Per ISA-RP37.2

Model No. V356A11/M034AD010AC

Serial No. 3647 (Z-AXIS)

PO No. ____________ Customer ____________

Calibration traceable to NIST thru Project No. 822/256889

ICP® ACCELEROMETER
with built-in electronics

Calibration procedure is in compliance with ISO 10012-1, and former MIL-STD-45662A and traceable to NIST.

CALIBRATION DATA

Voltage Sensitivity	10.74	mV/g
Transverse Sensitivity	3.5	%
Resonant Frequency	≥35	kHz
Output Bias Level	8.1	V
Time Constant	0.4	s

KEY SPECIFICATIONS

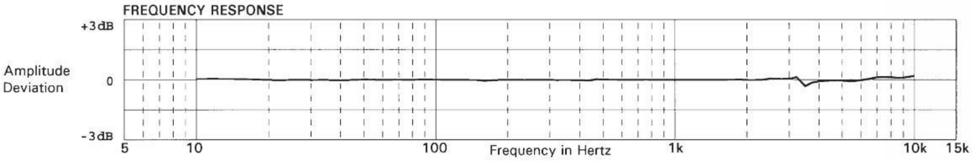
Range	500	±g
Resolution	0.002	g
Temp. Range	-65/+250	°F

METRIC CONVERSIONS:
ms^{-2} = 0.102 g
°C = 5/9 x (°F - 32)

Reference Freq.

Frequency Hz		10	15	30	50	100	300	500	1000	3000	5000	7000	10000		
Amplitude Deviation %		0.5	0.3	-0.2	0.1	0.0	-0.1	0.1	-0.1	0.5	-0.5	1.5	2.3		

FREQUENCY RESPONSE



PCB® Piezotronics, Inc. 3425 Walden Avenue Depew, NY 14043-2495 USA
716-684-0001

Calibrated by Martin Coleman
Date 03-11-1997

Appendix D

Table D.1 Mode description, natural frequency and mode shown in figure number for natural frequencies and modes according to Master's thesis model according to BRO 2004.

Description of mode shape	Natural frequency	Mode number	Figure number
Lateral movement of bridge deck	0.620	1	D.1
Vertical movement of bridge deck	1.386	2	D.2
Lateral movement of pylon	1.390	3	D.3
Lateral movement of bridge deck	1.441	4	D.4
Lateral movement of pylon	1.479	5	D.5
Vertical movement of bridge deck	1.899	6	D.6
Torsional movement of bridge deck	2.350	7	D.7
Lateral movement of bridge deck	2.559	8	D.8
Longitudinal movement of pylons	2.760	9	D.9
Longitudinal movement of pylons	2.786	10	D.10

Table D.2 Mode description, natural frequency and mode shown in figure number for natural frequencies and modes according to Master's thesis model according to Eurocode.

Description of mode shape	Natural frequency	Mode number	Figure number
Lateral movement of bridge deck	0.746	1	D.1
Lateral movement of pylon	1.539	2	D.3
Lateral movement of pylon	1.644	3	D.5
Vertical movement of bridge deck	1.648	4	D.2
Lateral movement of bridge deck	1.732	5	D.4
Vertical movement of bridge deck	2.285	6	D.6
Torsional movement of bridge deck	2.714	7	D.7
Lateral movement of bridge deck	3.057	8	D.8
Longitudinal movement of pylons	3.244	9	D.9
Longitudinal movement of pylons	3.274	10	D.10

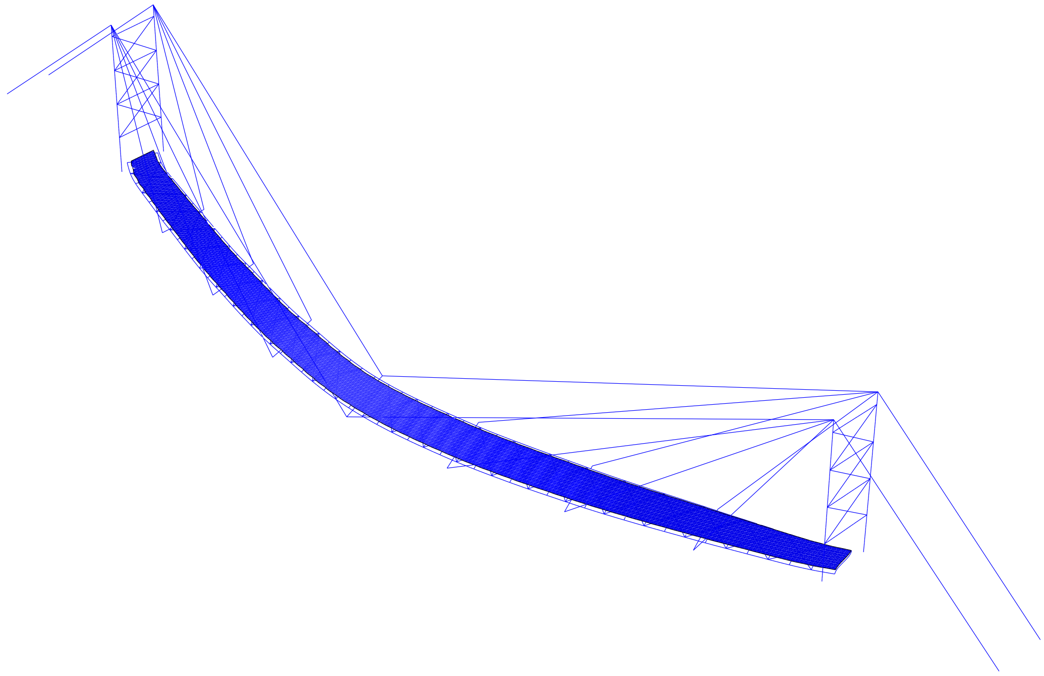


Figure D.1 First lateral mode of the bridge deck. The natural frequency from Master's thesis model according to BRO 2004 is 0.620 Hz and the natural frequency from Master's thesis model according to Eurocode is 0.746 Hz.

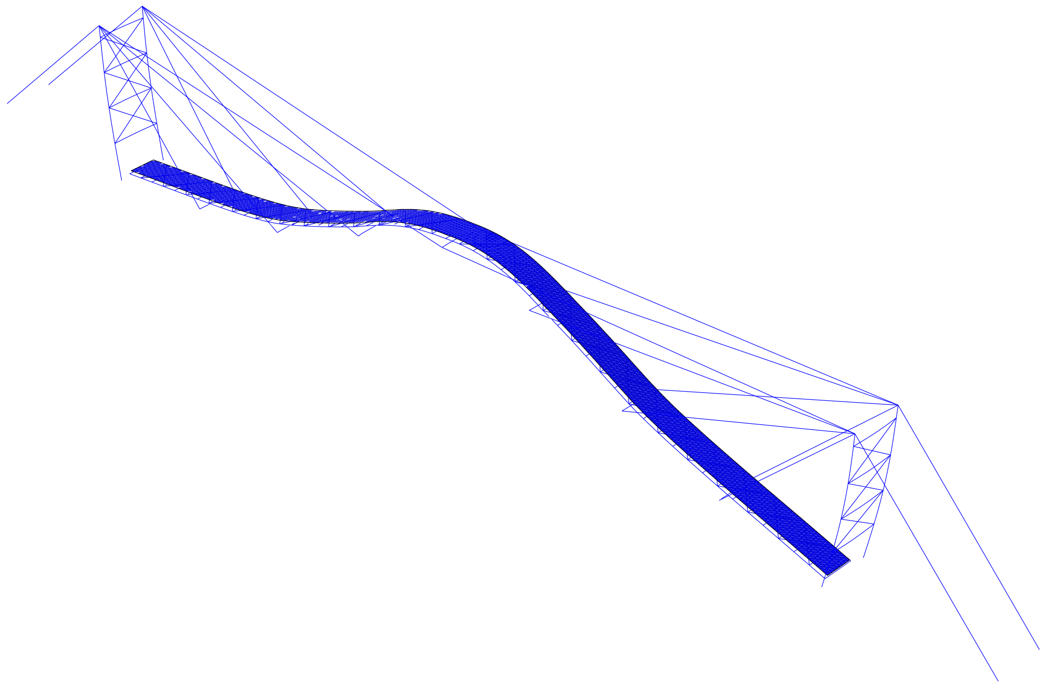


Figure D.2 First vertical mode of the bridge deck. The natural frequency from Master's thesis model according to BRO 2004 is 1.386 Hz and the natural frequency from Master's thesis model according to Eurocode is 1.648 Hz.

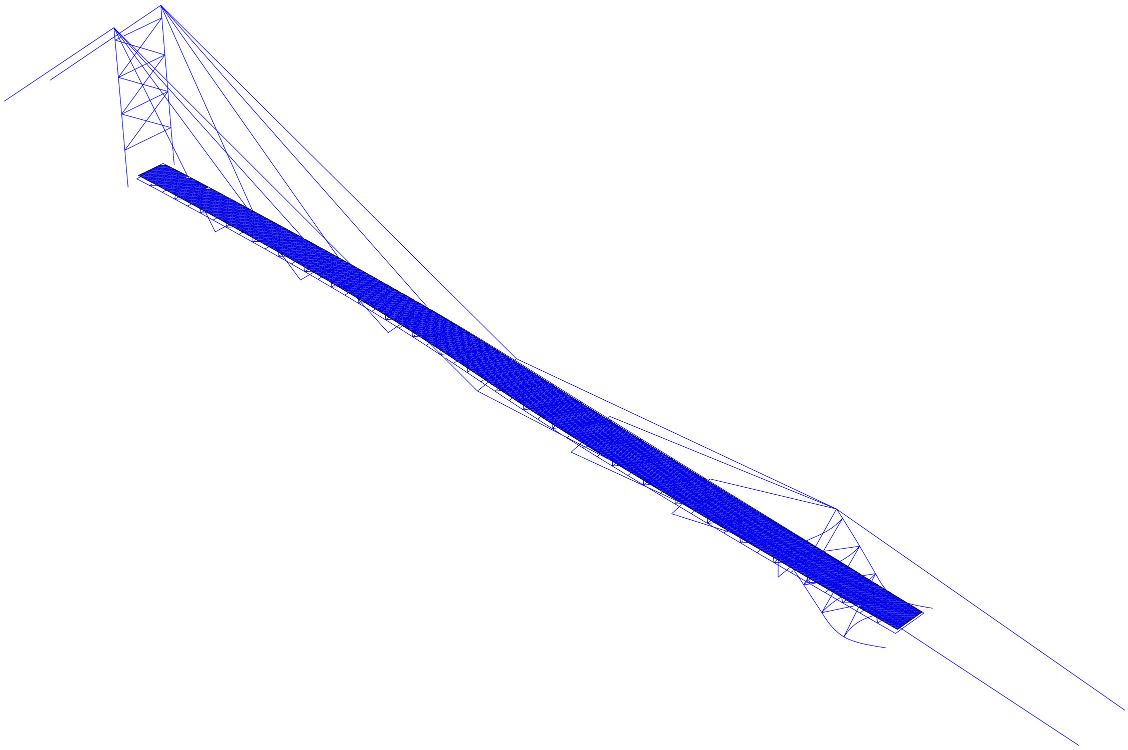


Figure D.3 *First lateral mode of the bridge pylon. The natural frequency from Master's thesis model according to BRO 2004 is 1.390 Hz and the natural frequency from Master's thesis model according to Eurocode is 1.539 Hz.*

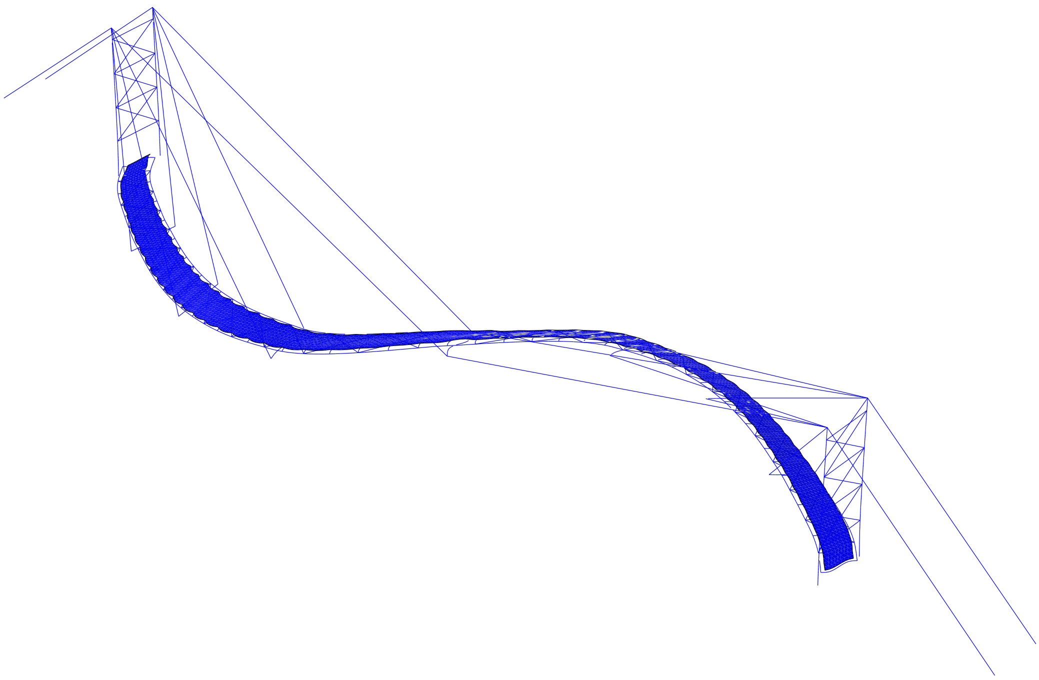


Figure D.4 *Second lateral mode of the bridge deck. The natural frequency from Master's thesis model according to BRO 2004 is 1.441 Hz and the natural frequency from Master's thesis model according to Eurocode is 1.732 Hz.*

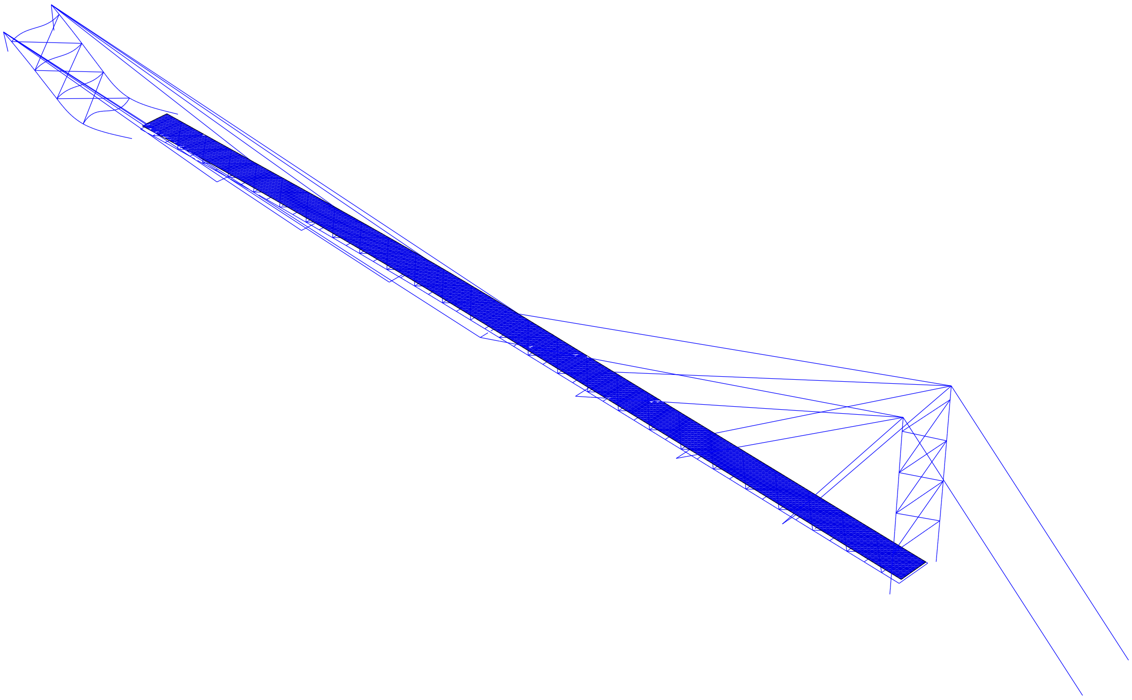


Figure D.5 Second lateral mode of the bridge pylon. The natural frequency from Master's thesis model according to BRO 2004 is 1.479 Hz and the natural frequency from Master's thesis model according to Eurocode is 1.644 Hz.

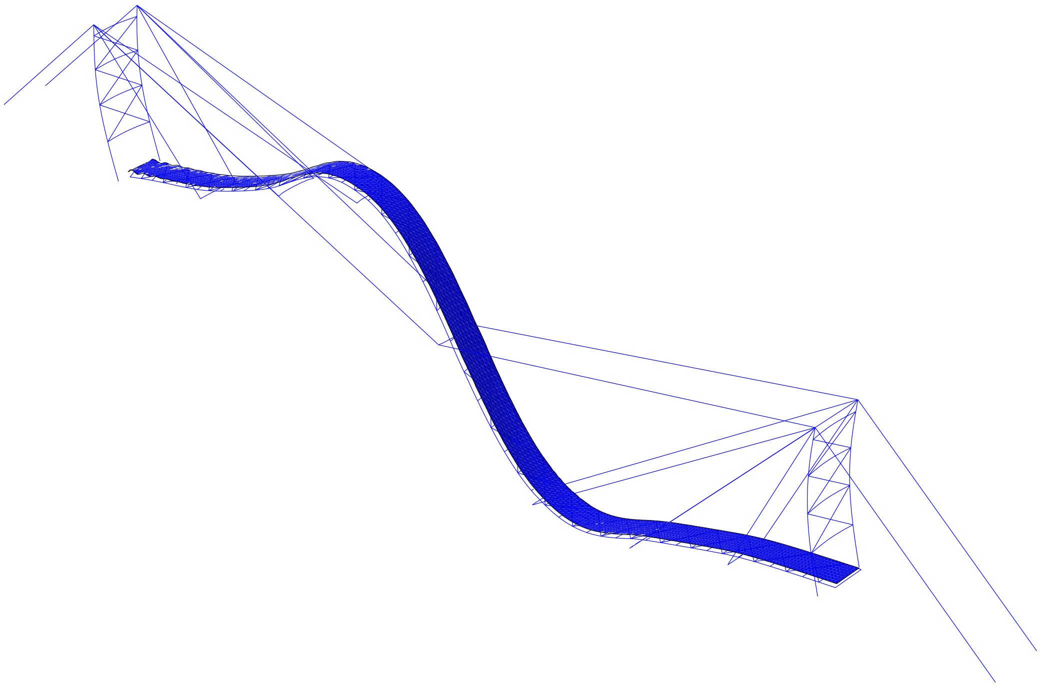


Figure D.6 Second vertical mode of the bridge deck. The natural frequency from Master's thesis model according to BRO 2004 is 1.899 Hz and the natural frequency from Master's thesis model according to Eurocode is 2.285 Hz.

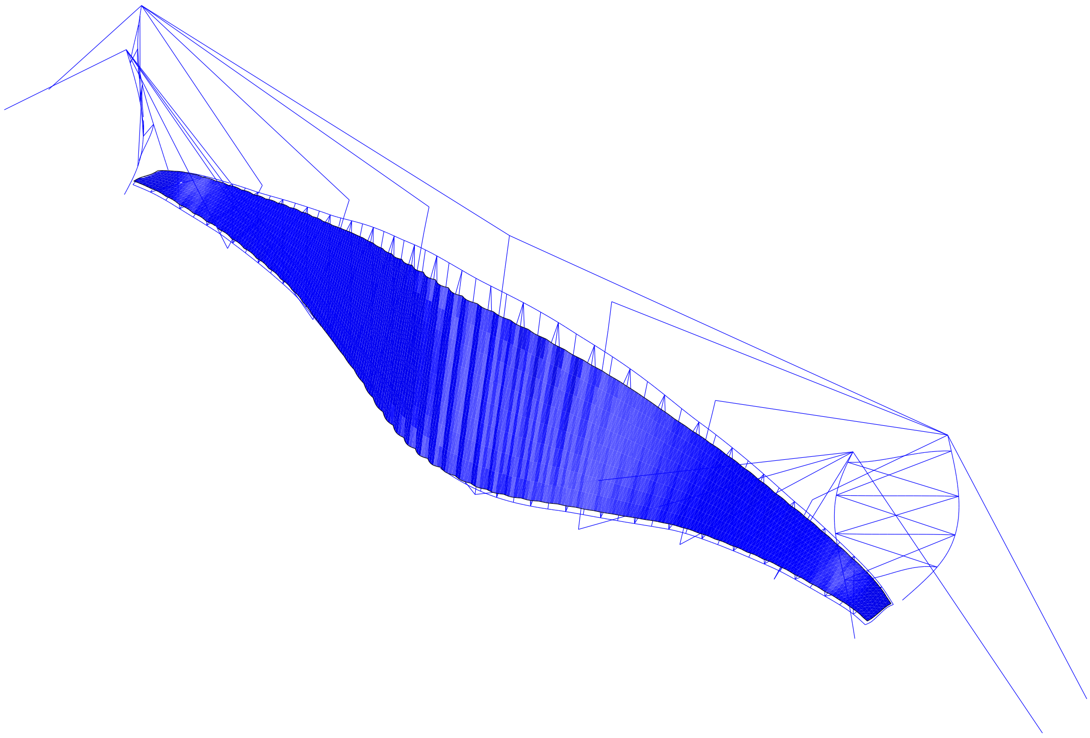


Figure D.7 *First torsional mode of the bridge deck. The natural frequency from Master's thesis model according to BRO 2004 is 2.350 Hz and the natural frequency from Master's thesis model according to Eurocode is 2.714 Hz.*

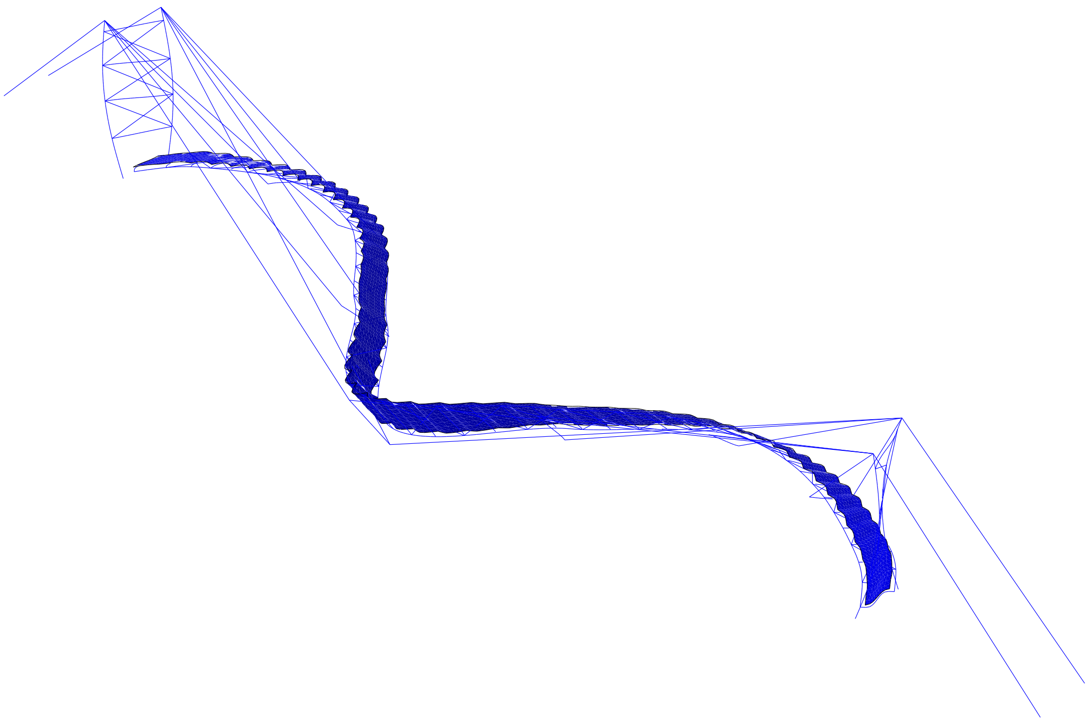


Figure D.8 *Third lateral mode of the bridge deck. The natural frequency from Master's thesis model according to BRO 2004 is 2.559 Hz and the natural frequency from Master's thesis model according to Eurocode is 3.057 Hz.*

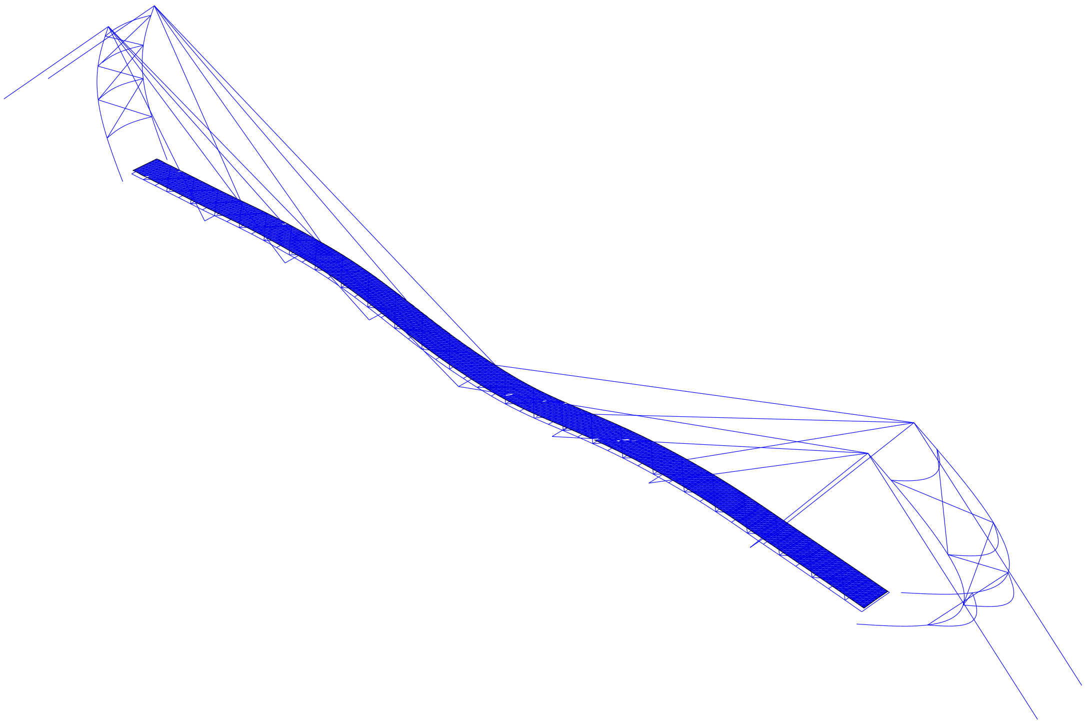


Figure D.9 *First longitudinal mode of the bridge pylons. The natural frequency from Master's thesis model according to BRO 2004 is 2.760 Hz and the natural frequency from Master's thesis model according to Eurocode is 3.244 Hz.*

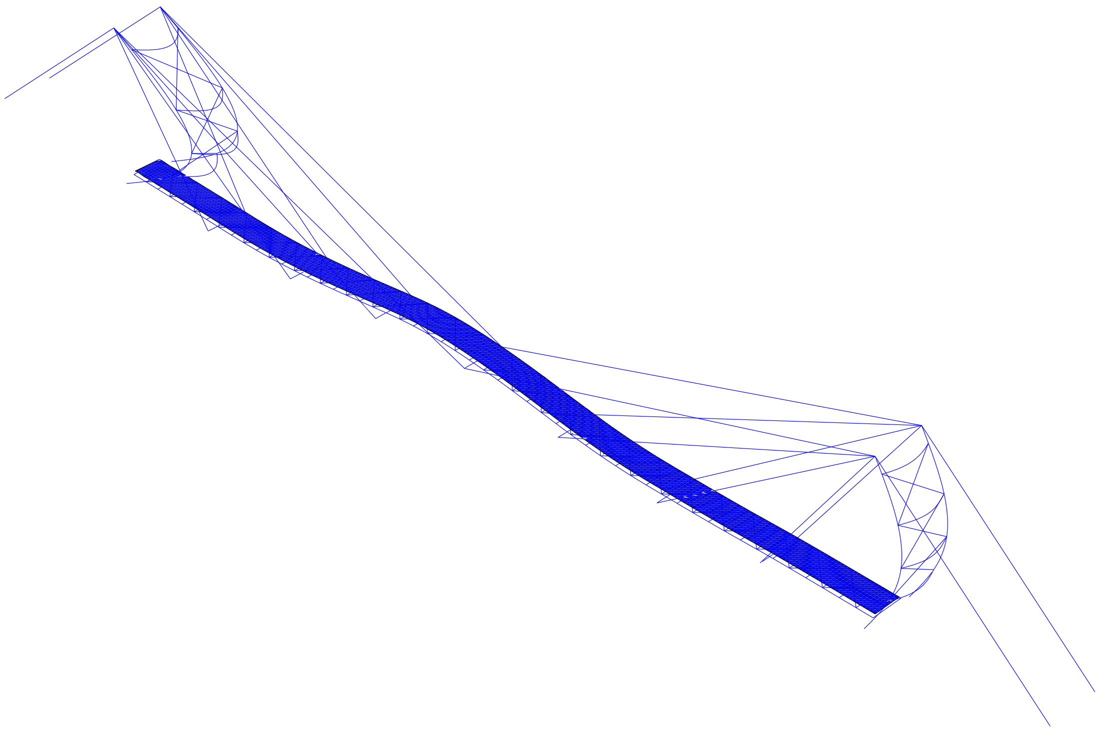


Figure D.10 *Second longitudinal mode of the bridge pylons. The natural frequency from Master's thesis model according to BRO 2004 is 2.786 Hz and the natural frequency from Master's thesis model according to Eurocode is 3.274 Hz.*

Appendix E

```
%% Calculation of damping factor
t = filtery.time ; % Time from filtered measurements
y = filtery.data ; % Accelerations from filtered measurements
sampleRate = 1/(t(2)-t(1)) ;
startTime  = 40  * sampleRate ; % Test dependent variable
endTime = 80 * sampleRate ; % Test dependent variable
upOrDown = sign(diff(y)) ;
maxPos = [upOrDown(1)<0 ; diff(upOrDown)<0 ; upOrDown(end)>0] ;
tops   = find(maxPos) ;
t_coord = [] ; % Coordinates in "time-direction"
a_coord = [] ; % Coordinates in "acceleration-direction"
for i = 1:length(tops)
if (tops(i) > startTime) && (tops(i) < endTime)
t_coord = [t_coord t(tops(i))] ;
a_coord = [a_coord y(tops(i))] ;
end
end
fit_damp = fit(t_coord',a_coord','exp1') ;
plot(t,y,'-r',t_coord,a_coord,'ok')
hold on
plot(fit_damp,'b')
damp_factor = 1/(2*pi) * log(fit_damp(t_coord(1)) / fit_damp(t_coord(2)))

%% Function to calculate Fast Fourier Transformation
function [] = FFTtransformation(y,t)
L = length(y);
Fs = L / t(end);
NFFT = 2^nextpow2(L);
Y = fft(y,NFFT)/L;
f = Fs/2*linspace(0,1,NFFT/2+1);
plot(f,2*abs(Y(1:NFFT/2+1)))
xlim([0 5])
xlabel('Frequency (Hz)')
ylabel('|Y(f)|')
```