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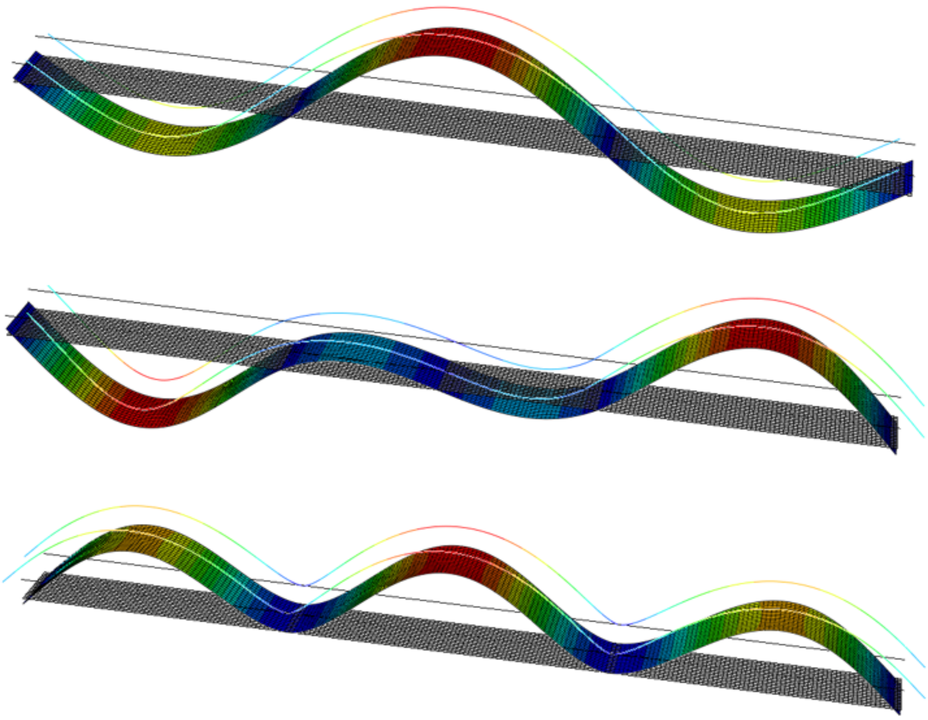

# Designing pedestrian stress-laminated timber bridges for multiple spans

## Parameters related to dynamic response

Master's thesis in Structural Engineering and Building Technology


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Master's Thesis ACEX30-19-36
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Cover: Oscillation modes of a three span bridge, modelled in Abaqus

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# Abstract

As the design process of bridges becomes more efficient and more slender bridges can be constructed, new design problems will arise. In light-weight constructions such as timber bridges, the dynamic response may be more crucial govern design and there may be need for more strict and well defined guidelines than before.

It would benefit the construction sector to make it effective to design more material efficient bridges in a environmental friendly material such as timber. This, since it makes it easier to answer to the increasing demand for a more sustainable development.

This thesis aims at providing a deeper understanding of the dynamic effects in stress laminated timber bridges. The goal was to develop the dynamic design process for SLT bridges by finding crucial, influencing parameters that affect the acceleration and dynamic behaviour. The studied parameters were, inter alia, span-to-span ratios, damping ratios and density changes.

The theoretical background was gathered by a literature review together with interviews in order to find which parameters that should be studied. The theory also provides a compilation of the dynamic design codes in America, Canadian and Europe. A parameterized script was created in Python that can be run through the software Brigade/PLUS in order to perform a frequency analysis and calculate the corresponding accelerations in the bridge. The script creates a model that was used to study what parameters the bridge is sensitive to.

The results showed that the accelerations correlated with multiple parameters and that a lot of improvements can be made in the design process.
Some of the main conclusions that can be draw from the result was that even though the critical damping ratio requires a lot of safety margin it can be increased from the 1%, that is used now, to 2%. This will make a tremendous difference when designing this type of bridge since it has a exponential correlation. Another main conclusion from this thesis is that further studies need to be made on the dynamic behaviour as the material properties are altered.

Keywords: SLT, Brigade/PLUS, Eurocode, Vibrations, Eigenfrequency, Damping

Dimensionera flerspans gång -och cykelbroar med tvärspända plattor
Parametrar relaterade till dynamisk respons
LINNEA AMUNDSSON OCH SABIHA FASTH
Instutitionen för Arkitektur och samhällsbyggnadsteknik
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# Sammanfattning

Eftersom designprocessen för broar blir mer och mer effektiv så kan slankare broar konstrueras. Detta resulterar i att det uppkommer nya problem i designprocessen. I lätta konstruktioner så som träbroar så kan den dynamiska responsen komma att bli mer avgörande, då frekvenserna blir lägre. Detta innebär att eventuellt mer väldefinierade och strikta riktlinjer krävs än i nuvarande design koder.

Det skulle gynna byggsektorn att göra det mindre komplicerat att utforma mer materialeffektiva broar i ett miljövänligt material så som trä. Detta för att göra det lättare att möta den ökande efterfrågan på en mer hållbar utveckling.

Syftet med detta masterarbete var att ge en djupare förståelse för den dynamiska effekten i tvärspända plattbroar samt att utveckla den dynamiska designprocessen genom att hitta avgörande parametrar som påverkar det dynamiska beteendet. Studerade parametrarna var bland annat spanförhållanden samt kritisk dämpning och densitetsförändringar.

En teoretisk bakgrund utfördes genom en litteraturstudie tillsammans med intervjuer för att lokalisera vilka parametrar som skulle studeras. Teorin sammanställer de dynamiska designkoderna i Amerika, Kanada och Europa. Efter teorikapitlet var konstuerat så skapades en kod för en parametriserad modell. Denna koden skapades i programmet Python och kan koden kan köras i programvaran Brigade / PLUS för att utföra en frekvensanalys och beräkna motsvarande accelerationer i bron för att se vilka parametrar bron är mest känslig för.

Resultaten visade att accelerationerna korrelerade med flera parametrar och att många förbättringar kan göras i designprocessen.

En av de viktigaste slutsatserna som kan dras från resultatet var att även om den kritiska dämpningen kräver en hel del säkerhetsmarginal så kan den ökas från 1 %, som används nu till 2 %. Detta kommer att göra en stor skillnad vid utformningen av denna typ av bro eftersom den har en exponentiell korrelation. En annan viktig slutsats från denna rapport är att ytterligare studier måste göras på det dynamiska beteendet då materialegenskaperna förändras.

Nyckelord: Brigade/PLUS, Eurocode, Vibration, Egenfrekvens, Dämpning

# Preface

This master thesis was created as a final assignment in our five year long journey that led us to our degree in civil engineering. In front of you is a report that summarizes six months of hard work where we tried to implement as much of the knowledge that we have assembled during these five years as possible.

During our work with this master thesis we encountered both prosperity but also a few setbacks. However, during these setbacks we always received help or guidance in the right direction. This is something that we will take with us in our future work as engineers. To never be afraid to ask for help and to take advantage of the skills that other people process.

Over these previous six months there have been many persons involved in our work and we have gotten a huge amount of help. For this we are very grateful and would like to thank everyone that made this thesis possible to complete.

We would like to thank our examiner, Robert Kliger and our supervisor, Robert Jockwer from Chalmers for providing us with a lot of tools to initiate this project. Your experience together with all the encouragement and valuable input made the project doable. We would also like to thank Peter Folkow and Morgan Johansson for providing a lot of input and knowledge from their areas of expertise even though it was not their job to help us.

We would specially like to thank Frida Gustavsson at COWI for the immediate feedback to all of our questions and for your guidance when we lost our way. Without Frida, this project would not have been completed. Additionally a huge thanks to all the fantastic people working at the division of Civil Structures at COWI for taking the time to help us.

In this report we have gotten the opportunity to apply a lot of the knowledge that we have collected during our years at Chalmers as well as learning a lot of new skills from our colleagues at COWI. We are very proud to be able to publish this thesis as a starting point for a more sustainable development in the construction sector and we hope that the work will be further developed in the future.

We hope that you will enjoy your reading.

Linnea Amundsson and Sabiha Fasth

Göteborg June 2019

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# Abbreviations

| | |
|---|---|
| **AASHTO** | American Association of State Highway and Transportation Officials |
| **AFGC** | French Association of Civil Engineering |
| **ASCE** | American Society of Civil Engineers |
| **BC** | Boundary conditions |
| **CHBDC** | Canadian Highway Bridge Design Code |
| **CLT** | Cross Laminated Timber |
| **CSA** | Canadian Standards Association |
| **EC** | Eurocode |
| **FEM** | Finite Element Method |
| **LM** | Load Model |
| **PSF** | Pounds per Square Foot |
| **PhD** | A degree awarded to people who have done advanced research into a particular subject |
| **Sétra** | Service d'Études Techniques des Routes et Autoroutes |
| **SLS** | Serviceability Limit States |
| **SLT** | Stress Laminated Timber |
| **ULS** | Ultimate Limit States |

# Notations

| | | |
|---|---|---|
| $c$ | Damping coefficient | [-] |
| $c_{cr}$ | Critical damping coefficient | [-] |
| $EI$ | Stiffness | [N/m$^2$] |
| $f$ | Frequency | [Hz] |
| $F_d$ | Damping force | [N] |
| $K$ | Span-to-Span factor | [-] |
| $k$ | Stiffness | [Pa] |
| $m$ | Mass | [kg] |
| $M$ | Total mass of a structure | [kg] |
| $n$ | Number of pedestrians | [-] |
| $W$ | Dead load of a structure | [N] |
| $x$ | Position | [m] |
| $x''$ | Response acceleration | [$m/s^2$] |

| | | |
|---|---|---|
| $\alpha$ | Acceleration | [m/s$^2$] |
| $\xi$ | Damping ratio | [-] |
| $\pi$ | = 3.1415... | [-] |
| $\rho$ | Density | [kg/m$^3$] |
| $\chi$ | Dynamic response factor | [-] |
| $\psi$ | Reduction factor | [-] |
| $\omega$ | Angular velocity | [rad/s] |

# 1
# Introduction

The environmental issues are currently a well discussed topic. In order to counteract the future negative effects, such as resource depletion, all sections of society must strive for change. One of the major issues that is of large significance in the construction sector is material choices. Currently there are three primarily used materials: concrete, steel and timber.

In bridge construction, timber is becoming a popular alternative to concrete and steel because of its diverse applicability and renewable properties. According to Eliasson (2017b), timber bridges are well suited for pedestrian and bicycle traffic due to its low self weight whilst a stronger material often is required for the larger and more highly utilized bridges. Due to the low self weight, pedestrian bridges in timber can have a more slender design than corresponding bridges in other materials where the self weight contributes to a large part of the load. Eliasson (2017b) writes that the most common type of pedestrian timber bridges are built with stress laminated timber (SLT) decks. An SLT bridge is made from a number of timber or glue-laminated beams positioned side by side and stressed together by using high-strength steel bars. Eliasson continues to describe that serviceability limit state (SLS) requirements, such as vibrations, can be the limiting design factor for these slender and low-weight bridges instead of ultimate limit state (ULS). Therefore the dynamic response becomes extra important as the span of the bridge increases. To make a timber bridge as optimally designed as possible with regard to dynamic response, it becomes relevant to consider the dynamic loads and eigenfrequencies of the structure.

It is unusual that the dynamic loads alone will lead to failure but they can nevertheless be determining in terms of function and comfort. Since the dynamic loads often do not contribute to a failure criteria, the limit values differ among different codes and the calculation methods are often insufficient or are simply not included in the code. Therefore it is necessary to study the dynamic design process in different countries to see how the process differ and how the differences affect the final design. According to Mårtensson (2016), the client determines how good the dynamic response should be in the construction but the standards provide general calculation methods as well as proposed limit values.

When designing bridges, finite element method (FEM) software is often used but as earlier mentioned there is no explicit computation method to calculate the dynamic effects. Therefore, it is common that the dynamic checks are evaluated in other programs in the post-processing. At COWI, the bridge design is made in the FEM software Brigade/-PLUS and the dynamic effects are tested in the structural analysis application Nastran. As a consequence the bridge has to be modelled in two different programs and it would therefore be beneficial to do the whole design process in the same program, Brigade/PLUS.

There are a few known parameters that affect the dynamic response of the bridge. For example the depth of the deck and the length of the span which consequently means a change in the geometrical stiffness of the deck. However, there are parameters that need to be further investigated to see how crucial they are for the design of the bridge as well as parameters that have not been investigated at all. Finding these crucial dynamic parameters, evaluate their correlation with each other and finally optimizing the final design of the bridge with regard to these parameters are important objectives that are treated in this thesis.

## 1.1 Purpose

The purpose of this master thesis was to make it easier and more attractive to design bridges in timber. Since timber is a eco-friendly material which is highly available in Sweden, it would be advantageous to be able to use it more in construction in order to reduce the environmental impact that the construction sector has. The purpose could be achieved by creating an accurate model that makes the dynamic process of timber bridge design easier and by presenting recommendations for a more optimized design process that will give an equal or possibly higher utilization rate.

The purpose was achieved by tackling the following objectives:

- To gain a deeper understanding of the dynamic effect and function of continuous pedestrian SLT bridges as well as a parametrization of the underlying design calculations.
- Challenge the current design calculation process in Eurocode (EC) by comparing different standards that are used. This, in order to make design less conservative and consequently more material efficient.
- To study parameters that could have an influence on the dynamic design.
- To study how large impact the known influencing parameters have on dynamic response and how do they correlate to each other
- Provide a starting point for further development of dynamic design of pedestrian bridges.

## 1.2 Limitations

This study is limited to continuous pedestrian bridges with three spans, a standard width of 4.0 m and a depth of 0.64 m will be used for the study in Brigade/PLUS.
The bridge was modeled as a linear, orthotropic plate and the study primarily looked at three vertical oscillation modes. See Figure 1.1 for the vertical oscillation modes that the three span bridge have. At COWI a majority of the recently constructed SLT pedestrian bridges have been three span. According to the Canadian highway bridge design code (CHBDC), there will be $n$ relevant oscillation modes for a $n$-span bridge. That is why

the limitation was chosen to primarily look at the first three modes for three span bridges.

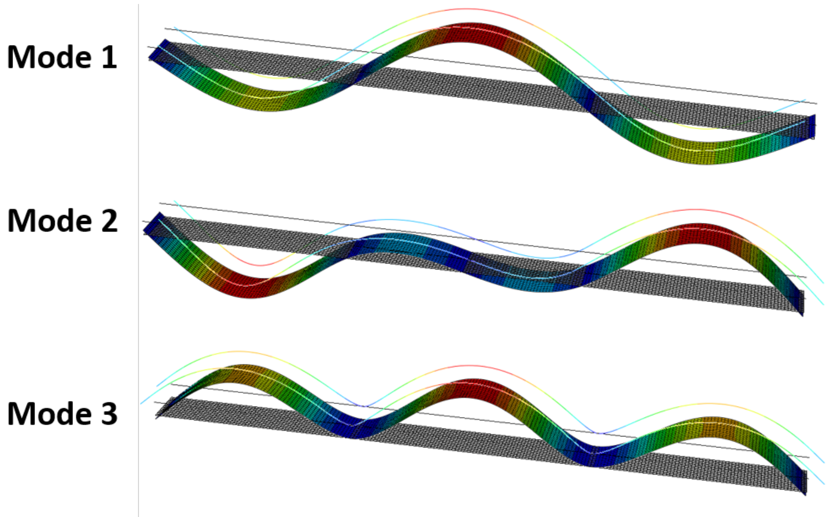


**Figure 1.1:** Vertical mode 1, 2 and 3 for a three span bridge with mid-span longer than equally long end-spans

These limitations were set to be able to study the behaviour of a common pedestrian SLT bridge. The behavior evaluation was made to see which parameters that are most relevant for the dynamic design process in a general SLT bridge. A check of wind induced oscillations in the transverse plane are not made. The bridge deck is assumed to be stiff and heavy enough in transverse horizontal direction to handle this types of load.

The report will only treat the dynamic effects and how these influence the final design of the bridge. The material properties and dimensions in the Python script will be parameterized so that the script can be used to design similar bridges with different input data.

## 1.3 Method

The project started with a literature study consisting of three parts. The first part is a theoretical background that explains and supports all assumptions made in the Python script. The python script builds the SLT bridge model as well as supports why the studied parameters were chosen. The second part consists of a compilation of previously executed studies as well as what further investigations that are needed for more accurate conclusions to be drawn. The third part treats different standards that are used today in different parts of the world to show how they work as well as what advantages and disadvantages they possess. In addition, interviews with engineers and professors were made to decide which parameters would be the most relevant to study.

After the theory was conducted the design process was performed. It entailed working in a finite element modelling program called Brigade/PLUS, an expansion of ABAQUS, which is adapted for modeling of bridges. An Excel input file was created and connected to the Python script. The script is built up by sections and all parameters are entered in the Excel input file. The script can be run through Brigade/PLUS and thereafter the output data will be printed in the Excel output file. Three existing SLT bridges have been constructed by COWI and were used to verify the script. The three bridges: Härlövsängaleden, Hasselfors and Vångavägen had been designed in Nastran, with the help of those design values a verification of the script could be done, both with regards to frequency analysis but also acceleration analysis. The script was thereafter parameterized in order to make it possible to use with different input data.

The finished script was used to enter different input data and change different parameters to see how this affected the acceleration and eigenfrequency of the bridge. The parameters that were chosen to be modified were based on the result of the literature study together with interviews. The result from the study was plotted to see correlations with the different parameters and then evaluated.

Finally, an evaluation was made based on the theory as well as the study and conclusions for an optimization of dynamic design were drawn.

# 2
# Theoretical background

The theoretical background treats the background information that is relevant for the report. A short description of SLT and vibrations in bridges are presented as well as information regarding different design codes. A general background to structural dynamics is also presented in order to provide the information needed to understand the question at issue and also how the model is established. In the end of this chapter a summary is presented with key findings of the theory together with useful input that was applied in the selection of parameters to study.

## 2.1 Stress laminated timber bridges

According to Eliasson (2017b), SLT bridge decks are a good selection for construction of short-and medium span bridges due to cost and performance aspects. An SLT bridge is made from a number of timber or glue-laminated beams positioned side by side in longitudinal direction and stressed together by using high-strength steel bars, see Figure 2.1. In Figure 2.2 definition of direction are visualized. Due to the friction between the beams that is caused by the pre-stressing, a concentrated load acting on one beam can be distributed onto the adjacent beams.

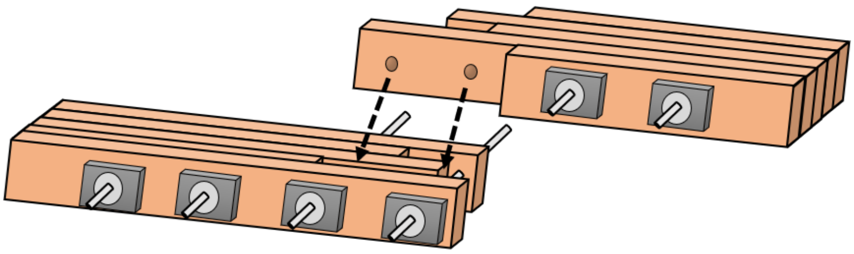

**Figure 2.1:** Cross-section of a small part of a SLT deck

One advantage of SLT decks is that bridge decks can be constructed as continuous decks and be adapted to the length required for the specific bridge. SLT decks can also be designed with cross-sections such as T-beam or box beam, in order to enable longer spans. Another option to increase the span is to include post-tensioning along the bridge axis.

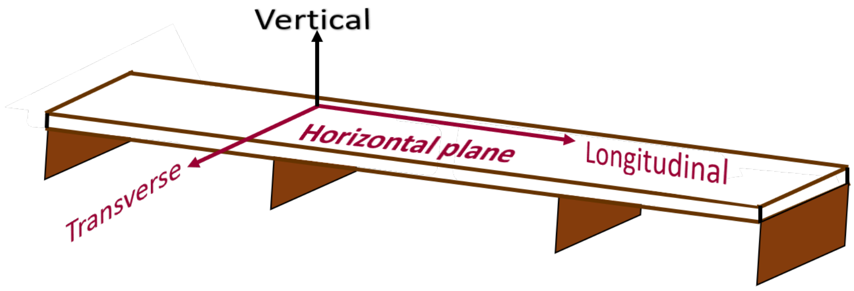


**Figure 2.2:** Vertical and horizontal (longitudinal and transverse) direction visualized on a bridge deck.

Crocetti (2014) states that there are many advantages to build bridges in SLT, specially in Sweden, in comparison to bridges in other materials. A few examples of advantages with SLT are:

- Short construction time and easy manufacturing/production.
- Benefits due to low cost and locally produced material
- Light-weight material with exceptional strength in relation to its density
- Sustainable material

Crocetti continues to explain that a bridge, made from a light-weight material is often very favourable in construction but it could add complication in terms of the dynamic response. The reason is that the low mass does not dampen vibrations as good as a heavier materials. Another complication with light-weight bridge decks is that the eigenfrequencies are lower, consequently they will be closer to pedestrians walking frequencies which might result in resonance, this will be further explained in Chapter 2.2.2.

Kliger et al. (2012) summarizes in their paper different research that has been made on timber constructions, primarily in northern countries. The research has a large focus on SLT decks and they highlight that dynamic design in serviceability limit state, SLS, will be a determining factor for bridges exposed to light traffic load and that the effects of dynamic damping and eigenfrequencies are of largest interest when performing the design. The authors also stated that since the EN standard has been modified to design for heavier traffic load, this might result in over-design for the pedestrian bridges.

Negrão (2016) addresses in his article another problem that can be observed in SLT decks, namely the loss of stiffness in the plate due to reduction of tension in the steel bars. This is a long term effect that is the result from stress relaxation and in some extent, creep. SLT decks often have the possibility to re-tension the steel bars after the bridge has been built. However, this is mostly done during the first years of the bridge's lifetime. Negrão writes that a 1-5% decrease in stiffness in the first year have been detected, provided that the steel bars are not re-tensioned it can lose up to 30% of its stiffness during the whole service life.

Crocetti et al. (2016) also lifts the matter of loss in pre-stressing bars in their article. They have found similar numbers on the amount of loss in the bars but state that as long as the loss is below 40% the deck will still have a adequate performance in the static point of view. They write that the anchorage system will perform in a different way if the deck is thin.

## 2.2 Dynamic effects in structures

There are mainly two effects that are observed in construction, static effect and dynamic effects. For bridges the static effects are well studied, however as mentioned in the introduction, the dynamic effect are not treated as much. In the following chapter a background to the concept of dynamics is presented along with the most important underlying calculations that are required for this thesis.

### 2.2.1 General

In a report by Boniface et al. (2006) it is explained that pedestrian load is very low and would not make a significant impact on a massive and stiff bridge. However, due to developed technology and building methods, bridges get more slender and material efficient. This results in stronger bridges with lower self weight which in turn leads to further requirements on the dynamic performance.

In a report written by Harish (2019) it is explained that there are several known cases when structures have failed due to vibrations, when so called resonance has occurred. For example, Tacoma bridge is a known case where the bridge was exposed to vigorous wind which caused the deck to oscillate. The frecuency of the oscillations reached the eigenfrequency (also called resonance frequency), which ended in collapse of the bridge. Another case where resonance has given severe consequences is the Millennium Bridge in London. Strogatz (2005) explains that when the bridge got crowded and the pedestrians started walking in a synchronized pattern a transverse horizontal sway of the bridge was initiated. This was due to the so called lock-in effect which is further explained in Chapter 2.3.3. The incident occurred because the slender bridges eigenfrequency was close to the one of the humans walking.

Ussher et al. (2017) discusses what parameters that affect dynamic behaviour in their article. They write that as the ratio between stiffness and mass increases it becomes harder to handle the dynamic loads. Further on they list the main known influencing parameters for these types of constructions.

- Support condition
- Boundary condition
- Span lengths
- How many persons that are walking on the structure (Mode shape)
- Modal frequency (associated with the mode shapes)
- Damping ratio

Ussher et al. conclude that these parameters also will correlate with each other which makes a general approach very hard. In the current situation a simulation needs to be made on each specific construction to see how it will handle the dynamic loads. The article is a study on cross laminated timber (CLT) floors but the authors emphasises that many of these parameters can be applied to all slender, light-weight slab systems. Therefore these parameters can be applied to SLT bridge decks as well due to the plate like behaviour.

### 2.2.2 Oscillation and Frequency

Sjörs & Bager (2019) describes frequency as the amount of oscillations per second which is measured in the unit Hertz (Hz). To calculate the frequency, one fixes a time interval, counts the number of events and divides this number by the length of the time interval.

Vigran (2008) explains in his book that structures have so called eigenfrequencies. In this frequency, the structure has a tendency to swing with greater amplitude than in other frequencies. This is because in the eigenfrequency, the structure can much easier store vibrational energy. He continues to explain that you can excite a structure to a higher frequency with an external force but damping will make the structure go back to its own eigenfrequency if you do not continue to excite it. The eigenfrequency for a structure is defined by the mass, span and stiffness (SS-EN 1995-1-1). A simplified equation that describes how to calculate the $n$:th frequency mode is presented below.

$$f_n = \frac{n^2 \cdot \pi}{2 \cdot L^2} \cdot \sqrt{\frac{E \cdot I}{\delta \cdot S}} \tag{2.1}$$

where $\delta \cdot S$ is the linear density of the bridge, including loads such as pedestrians. EI is the cross-sectional stiffness and L is the length. A complete notation declaration can be found in the beginning of this report.
Sjörs & Bager fortifies what Vigran said about eigenfrequenicies. They continue to describe that a system can, if it gets exposed to external excitation with a frequency that corresponds to the eigenfrequency of the structure, start to oscillate with higher amplitude and acceleration. This is the so called resonance frequency.
Resonance is a phenomenon which occurs when a system is able to store and easily transfer energy between different forms, for instance between kinetic energy and potential energy.

As one can imagine, a lighter structure will more easily start to oscillate, hence they have a lower resonance frequency. The reason why pedestrian bridges are more prone to be in resonance is that the low resonance frequency of the bridge approaches the same frequency as a pedestrian walking frequency. In the same manner as described about the Millennium Bridge in London.

According to Boniface et al. (2006) the resonance frequency can be expressed as:

$$f_R = \frac{\omega_R}{2 \cdot \pi} \tag{2.2}$$

Where

$$\omega_R = \omega_0 \cdot \sqrt{1 - 2 \cdot \xi^2} \tag{2.3}$$

$$\omega_0 = \sqrt{\frac{k}{m}} = 2 \cdot \pi \cdot f_0 \tag{2.4}$$

Radlert & Åkerblom (2015) explains that usually only a few of the resonance frequencies are of interest in a dynamic analysis, generally the lower frequencies. An example of when this occurred is, as earlier mentioned, the Tacoma bridge which failed due to resonance when the excitation frequency, in this case the wind, matched with the eigenfrequency of the bridge. Therefore, using the highest excitation load might not create the worst dynamic case, a lower load can be decisive if it has a frequency closer to the bridges eigenfrequency.

Each eigenfrequency is, according to Radlert & Åkerblom, connected to a given oscillation form, a so-called modal form. The modal form is the displacement shape that the system vibrates with.

### 2.2.3 Damping

In dynamics, the effect of damping is very important. Ansell & Svärd (2014) explains that one way to counteract resonance is to implement damping in the system. Damping will lead to the resonance decreasing, as seen in Figure 2.3, instead of getting a higher and higher amplitude which would be the case without damping.

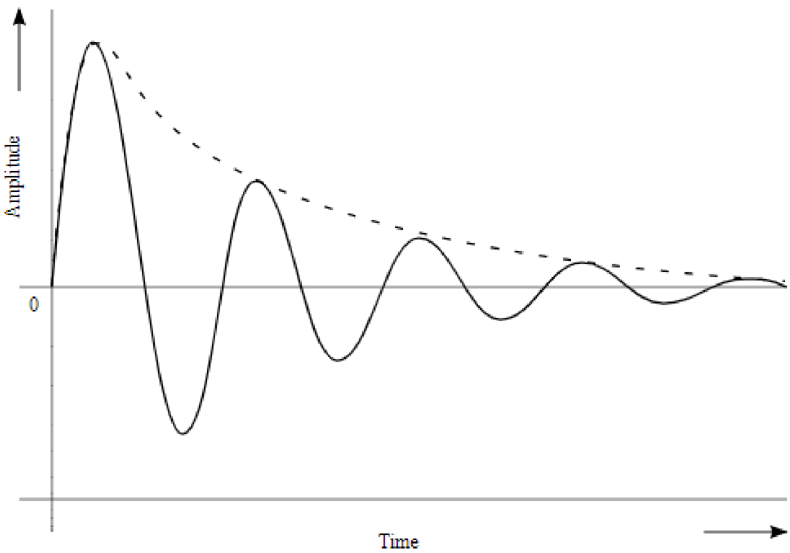


**Figure 2.3:** Damping effect on oscillation (Wikipedia, 2013) CC BY-SA 3.0

Labonnote (2012) state in her PhD thesis that damping has a greater impact on how we perceive vibration compared to both the frequency and the magnitude of the vibration. She continues in her state of the art to explain that damping can be either:

- Viscous
- Hysteretic
- Proportional
- Viscous Equivalent

Pousette (2001) writes in her report that for timber a viscous damping is assumed and even though damping is often non linear, a linear model can be applied if the dampening as well as the oscillation is relatively small. She also endorse Labonnote's statement that damping is very important in lighter structures such as timber bridges, specially in bridges with longer spans.

Pousette also highlights a problem when modelling damping in these types of structures. Full scale testing has been made on completed constructions which indicates that the damping is a lot higher in real life than what is used in design codes. She thinks that this is due to the protecting layer on top of the bridge deck as well as the railings on the sides. Currently these 2 factors are neglected when calculating or assuming the structures damping.

To calculate the structures damping, certain parameters must be known. The critical damping coefficient $c_{cr}$ that represents when the structure goes from still to oscillating, the viscous damping coefficient $c$ that is generally dependent on the angular eigenfrequency $\omega$ when the structure is undamped. Finally, the damping effect is also dependent on the mass of the structure $m$

The damping force, $F_d$, is dependent on damping coefficient, $c$, and the velocity response of the structur, $x'$, from the excitation force. It is expressed as:

$$F_d = c \cdot x' \tag{2.5}$$

where

$$c = 2 \cdot \xi \cdot m \cdot \omega \tag{2.6}$$

With the damping ratio and critical damping expressed as:

$$\xi = \frac{c}{c_{cr}} \tag{2.7}$$

$$c_{cr} = 2 \cdot m \cdot \omega \tag{2.8}$$

In these equations it is shown that an inaccurate value of the damping cause a large problem for the design values. This results in a higher acceleration, which is used to determine the dynamic response of the structure.
In a report by Radlert & Åkerblom (2015) they show that in some cases the damping can be up to 10 times higher in reality than what was calculated according to code. This can, as is shown in the equations, give design values that are larger than actually required. In EC a value of $\xi$= 0.01 is always assumed for timber decks, it is a fixed value that does not consider different geometries or masses. (SS-EN 1995-2:2004, 2004)

## 2.3 Dynamic loads

Loads can be of two types, they can be permanent load, for example self-weight, or varying loads which per definition means a load that is variable in time and space. According to the technical guide by Boniface et al. (2006), varying loads can then be static, for example snow load which is static during a period and then is absent for a period. They can also be time dependent, for example a pedestrian who moves. Dynamic loads are time varying loads and can be divided into four categories:

- Harmonic or pure sinusoidal loads.
- Periodically repeated loads, repeated at regular time intervals (Periods).
- Loads applied randomly, showing arbitrary variations in, for example intensity, time, direction etc.
- Pulsing loads corresponding to very brief load times.

This thesis treat pedestrian, time-varying loads that, according to the guide may be treated as a "periodic load".

### 2.3.1 Dynamic effects caused by vertical loads

In the guide by Eliasson (2017a), it is stated that the vertical loads will primarily consist of human induced loads on pedestrian bridge. There are many variations on rhythmic movements created by humans, which causes great variation in dynamic loads. Activity that generates synchronized rhythmic movements are particularly problematic, for example when several people march in pace or performs gymnastics. When synchronized movements occur for 20 seconds or longer, periodic loads may occur causing stationary structural vibrations.

The activity and load scenario that will be expected to occur on the bridge has to be considered when designing the bridge. Forces associated with a pedestrian bridge are likely to be loads created by people either walking or running. The two load cases differ in time and amplitude, runners have a shorter contact time but the force is higher which will give the worst case scenario.

The vertical forces created by walking and running have been measured and two peaks for both cases can be seen where the first peak is due to heel strike contact and the second peak is due to toe-lift off contact. In the remade Figure 2.4 the force of an individual foot impact is shown as a graph, the dashed line shows the next step.

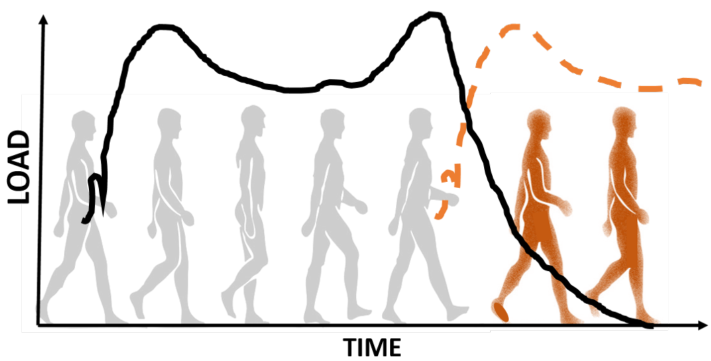


**Figure 2.4:** Visual description to horizontal load created when walking (Solid line symbolize right footstep and dashed line left footstep)

As stated in the introductory chapter, it is complicated to state how one perceive a certain type of vibration. The sensitivity towards low frequency vibrations are individually experienced and varies with regard to situation, for example if the person is seated, walking or resting. According to Mårtensson (2016) humans sense low frequency motions in three different ways:

- In form of **accelerations** - felt by the balance organs in the body.
- As **visual cues** - moves in relation to surrounding.
- As **audio cues** – noise crated by motion of the structure.

Structural frequencies in the range of 4 to 8 Hz are undesirable since this is the range of vibration for human bodies and it is also the range where humans are the most sensitive to vibrations. This means that humans will feel an uncomfortable feeling if they are walking or standing on a structure that vibrates in this frequency range.

### 2.3.2 Dynamic effects caused by horizontal loads and components

In this study, horizontal forces are not considered, as mentioned in the limitations chapter. The reason why this can be assumed is explained by Lidelöw (2016) who writes that in SLT decks it can be assumed that the glulam girders acts as a plate and will thus be stiffened against the horizontal loads in the transverse direction - all though not in the vertical direction. See figure 2.2. This means that forces moving transverse the bridge deck, for example wind loads, does not need to be considered.

Even though horizontal loads are not considered, there are still horizontal components from the vertical loads due to the horizontal movement that the person walking will have. Dyken (2017) explains that when designing a bridge for the horizontal force components, the force distribution in the deck, which is determined by the span to width ratio, have to be considered. For pedestrian bridges with a ratio higher than 4, a linear stress distribution can be used.

A horizontal bending mode need to be considered in dynamics if the frequency is below the limit value, which is specified by Eurocode to be $f_{horizontal} < 2.5$ Hz. According to Figure 4.2 the horizontal component will not need to be considered for the type of bridges that are studied in this thesis. In order for this parameter to be crucial, the supports has to be very weak.

### 2.3.3 The Lock-in effect

The mid point of a person varies in both vertical and horizontal direction when walking. According to Heinemeyer (2008), people in general are more sensitive to lateral vibrations than to vertical vibrations. The explanation is that the vertical vibrations are absorbed by legs and joints while lateral vibrations cannot be dampened by the body. Consequently, it has been observed that that crowds naturally synchronize in lateral direction but not in vertical directions. This synchronization is the so called lock-in effect.

The lock-in effect may occur in a structure that is allowed to vibrating laterally. A person walking will automatically move its center of gravity to increase stability (balance) by swaying with the lateral vibrating bridge displacement, see Figure 2.5. To move the center of gravity in time with the vibrations humans tends to walk with double the vibration frequency. This small additional horizontal force created by the human adds energy to the structure. Therefore, a crowd can easily start to walk in a synchronized pattern which will cause a large additional force that exposes the bridge to large vibrations.

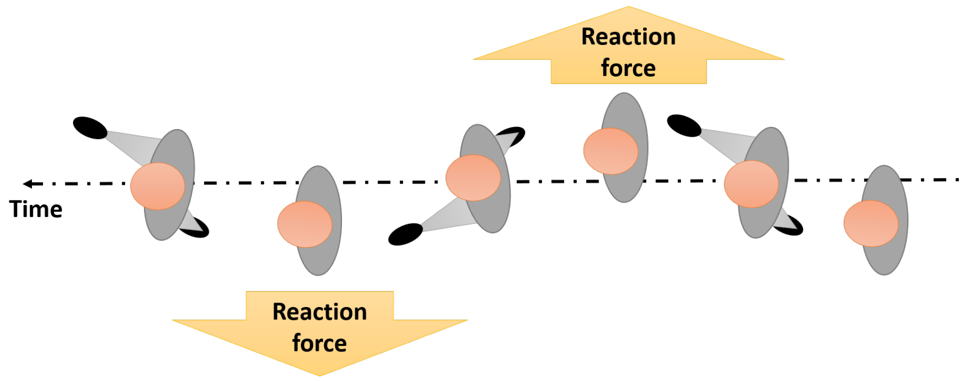


**Figure 2.5:** Visual description of lateral walking force, top view

## 2.4 Frequency response analysis

There are multiple ways to solve dynamic response equations for an arbitrary excitation. Géradin & Rixen (2014) explains that depending on the modes, different methods will be more or less advantageous to use. In general there are two different ways to perform frequency response analysis; a mode based steady-state dynamic analysis or a direct-solution steady state dynamic analysis. The mode based method is less accurate but more time efficient than a direct-solution.

In Abaqus Analysis User's Manual (2019) it is stated that the main difference between the analysis methods is that direct-solution produce dynamic loads and run the analysis for all frequencies in a selected frequency range while modal based analysis only run the analysis for the eigenfrequencies of the studied structure. For the modal analysis the size of the dynamic response equations are much smaller thus making the analysis run much faster. Direct-solution should be used for structures where you cannot calculate the eigenfrequencies, for example structures with non symmetrical stiffness, viscoelastic materials etc.

The result from the analysis is the steady-state acceleration response of a system due to harmonic excitation at a given frequency range. For the studies that are performed in this report the excitation is in form of pedestrians walking and the frequency range is from 0-5 Hz. Thus, instead of applying loads that are frequency dependent, the loads are calculated based on the eigenfrequencies. Then, the loads can be applied in a frequency band from 0-5 Hz. The peaks in acceleration will be when the load is applied in the same frequency as the eigenfrequency of the bridge, as you can see in Figure 2.6.

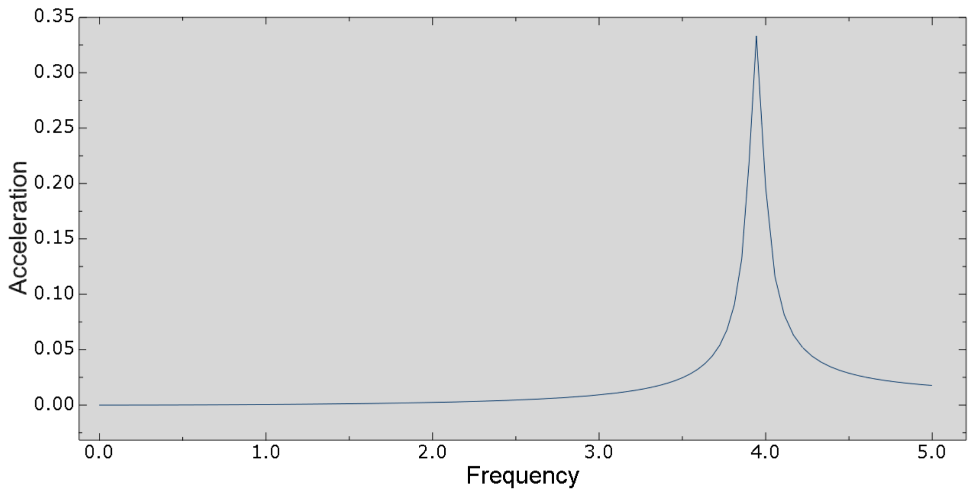


**Figure 2.6:** Example of resulting max acceleration when load is applied in a frequency band from 0-5 Hz where the eigenfrequency is 3.95 Hz

## 2.5 Influencing parameters and key findings of theoretical background

In this section, the most relevant parameters to study are summarized based on the theoretical chapter together with input from professors and engineers.

As mentioned previously, there are a few already known parameters that affect the dynamic response of a pedestrian SLT bridge. Ussher et al. stated that the width and length of the span together with the ratio between these parameters have a large influence on the dynamic effect. This, due to the fact that a change of these parameters will affect both the mass and the stiffness of the bridge . S. Lindén (personal communication, 04-03-2019) is head of the Construction section in Civil Structure department at COWI and he explained that geometrical values such as width of the bridge deck or the span often are either predefined by Trafikverket or chosen based on the conditions on site. These values might therefore be hard to influence and that is why it is often the thickness of the deck that is changed. However, he said that the placing of the supports might be altered partially as long as it coincide with the sites prerequisites.
R. Kliger (personal communication, 04-03-2019) is a professor at the Division of Structural Engineering at Chalmers and he added that it still might be of use to study how long these types of bridges can be made while still having adequate dynamic performance. When studying how long these bridges can be made it is also interesting to see what the ratio between the spans are and if that ratio makes a difference for the dynamic response. When doing this study from a dynamic point of view it is however still crucial to satisfy the deflection demand which is $\delta$<l/200 for pedestrian bridges, according to Table 7.1 in SS-EN1995-2:2004. The corresponding value is l/400 according to the Canadian Standards Associations (CSA) standard.

As presented in Chapter 2.2 the mass to stiffness ratio will have a massive impact on the dynamic response, specially as more slim and material efficient constructions are designed. Therefore, the thickness to span ratio of the deck will be of interest.

The damping parameter has been studied a lot which is presented in this chapter but also further down in the literature review. Both Pousette and Labonnote agree that the damping ratio is often set very low compared to the real value. However, they also state that since you cannot calculate the damping ratio in advance you have put a low value to be on the safe side. This is also confirmed by Jutila.
Another stiffness parameter that is well discussed in the static bridge design and presented in Chapter 2.1 is the loss of stiffness in the anchorage system from the pre-tensioning. As Crocetti et al. stated, a loss of stiffness up to 40% will still provide for good static performance. However, the effects on the dynamic response is not mentioned. He also says that this loss of stiffness will have extra effect in thinner decks which is often the case with pedestrian brides. Therefore, it should be studied what the dynamic effect will be due to this stiffness reduction.

There are also a few other parameters that are unstudied with regards to dynamic response that might be of interest to this study, for example how railings affect the dynamic response. For these parameters it is mostly important to see if they even have a correlation with the dynamic response, since no studies have been made on this. If it turns out that they have that, a more thorough study on this should be implemented.

In the discussion with Kliger, he said that the density parameter in timber will be of interest since the values that are used for density today have a high margin of error. When setting the density, the contractors include extra weight of steel bars etc and ensures that the value is on the safe side. The design value for density that is used at COWI is 600 $kg/m^3$ i.e. higher than expected and very conservative. Therefore, it would be of interest to see what effect it might be to chose a lower density than 600 $kg/m^3$ to see if and how this parameter correlates with the dynamic response.

Kliger states that the moisture content is of less importance for strength but it will however affect the stiffness of the timber. He says that in practise the variations in moister content is counteracted by large cross-sections as well as the coating material. Therefore, the moister content in the timber is of less importance but, as is also mentioned earlier, the actual stiffness is a influencing parameter that is interesting to study.

The coating material however, most often asphalt, will have a great influence on the damping that the bridge possesses together with the surrounding temperature as shown later in Chapter 3.3 in the literature review.

Another parameter that both Kliger and Lindén said might be of interest is the damping in the supports. How much damping do the rotational system posses when they are prescribed with different constraints? Damping in rotational system has been studied but no studies on how it affects the dynamic response has been found and might therefore be a good parameter to include in the analysis.

To summarize, parameters that are of interest are:
- Span-to-depth ratio
- Span-to-width ratio
- Span-to-span ratio

- Placing of supports
- Boundary condition
- Different aspects of material stiffness
- Stiffness reduction
- Density
- Different aspects of damping
- How different load cases affect the result. Note that the highest load is not necessarily the worst load.

# 3

# Literature review of previous studies

In this chapter you will find a compilation of the most important studies that has been made and are relevant to this thesis. In the end of this chapter, a summary is presented with the key findings and important information that was useful as a background for the model and to the parameter study.

## 3.1 Vibrations in pedestrian timber bridges

Jansson & Svensson (2012) made a master thesis about the dynamic response in Älvsbackabron when it was exposed to pedestrians walking on the bridge. The thesis made a comparison mainly between two different standards that have been or are being used in bridge design in Sweden, namely BRO 2004 and EC 5. They modeled the bridge according to both standards respectively to see how the results differed. The modelling was made in Brigade/PLUS. The resulting values from the models was also compared to physical measurements of the actual bridge. The purpose of the study was to see the difference between the standards and also which standard that could most correctly correspond to the real bridges behaviour. The study has a lot of similarities to the study being done in this thesis, which also will involve comparisons between different international standards and the use of the program Brigade/PLUS. Their result showed that EC was able to produce a more accurate model of the bridge than BRO 2004 did since it takes a larger amount of parameters, such as lateral vibrations etc. into count. Another important discovery in Jansson & Svenssons thesis was that the critical damping in the bridge was twice as large as the value used in the design code.

In a similar master thesis, Eriksson & Pagander Tysnes (2013) highlights the importance of dynamic design of light-weight structures. They performed a study of three different light-weight bridges according to four different design codes: Sétra, ISO 10137, EC 5 and EC with the UK national annex. They did a comparison to see how well the calculation of peak vertical acceleration corresponds to reality. The result of the comparison showed that Sétra and ISO 10137 had a much greater correspondence to real life.

Both reports are done in a small scale, but the result shows that there is a large difference between codes, both for calculations and for the correspondence to the real life value. Due to this difference, it is of great importance that codes are evaluated and developed to see which ones suits the current construction situation that is mentioned in Chapter 2.1.

In this thesis, Sétra will be used as design code. This is due to the fact that it has great correspondence to reality, as shown in the thesis written by Eriksson & Pagander Tysnes (2013) but also since that is the design code used at COWI.

In a lecture by A.Talja and H.Lilja (13-04-2017), the testing of five pedestrian bridges was presented. The test was made with the purpose to develop the Finnish national annex of EC and to see what might be critical in design. The bridges were all made of SLT and they were all single span. The spans-to-depth ratio was between 24-31, density was set to 400-500 $kg/m^3$ and some of the decks were covered with asphalt. The variation in first eigenfrequency was 2.5-4.5 Hz and the critical damping ratio that was found by testing varied between 2.5-4%. A conclusion they made after testing is that the damping value can be set to 2-3% and still provide for a safe bridge. After they compared the hand calculations of eigenfrequencies with the measured values they could state that the margin of error when can be up to 15% due to fluctuations in material properties.

Talja and Lilja continues with explaining that when studies on the perceived discomfort when the bridges were excited was made, it showed that the feeling of vibration was registered at accelerations from 0.5 $m/s^2$ and up. Large discomfort was perceived from 1 $m/s^2$ and up. However, they found a that if the the people were moving across the bridge they would not register any vibrations until 1.5 $m/s^2$. These tests supports what Mårtensson said about the complexity in how vibrations are perceived.

What the performed study also established was that a person running is always a worst case scenario for pedestrian bridges, even when compared to multiple persons walking out of sync.

## 3.2 Comparative Analysis of Design Codes

In the article by Wacker & Groenier (2010) a comparative analysis was presented of the national codes in the United States, Canada and Europe. Different parameters related to timber bridges, such as design for bending, shear and loads, were studied. There were both similarities and differences between the codes. All three codes considered ULS and SLS. The codes used the same structural dynamic equations but the primary difference was the use of adjustment factors and specified strength values. For example, the load combinations differed between the codes which affected the result in multiple ways. Regarding the U.S. and the Canadian standard, the differences in load combinations did not affect the final design load remarkably. EC on the other hand got two to tree times higher bending moment and shear effects than the U.S. and the Canadian codes gets. A comparative design of a glulam beam was made and the result showed that the largest size was required by EC while the smallest beam size was required by the Canadian code.

Conclusions that can be drawn from Wacker & Groenier (2010) are that EC design is on the "safe side" and is much more conservative, which leads to higher material consumption and larger costs than actually required. The conclusion is supported by Nowak et al. (2001) where the three different codes, Spanish Norma IAP-98 (1998), ENV 1991-3 EC 1 (1994) and American Association of State Highway and Transportation Officials, so called AASHTO LRFD (1998), were compared by designing pre-stressed concrete bridge girders. In their report, conclusions were drawn that EC was the most conservative while the U.S. standard was most permissive.

## 3.3 Influence of protecting layers on vibration and damping

Since timber is a sensitive material with regards to external, environmental impact it requires a protecting layer in order to provide for a long service life. The material of the protecting layer is often asphalt and in a paper written by Feltrin et al. (2011) the damping effect of asphalt was studied together with how it behaved with varied temperature. As earlier mentioned in Chapter 2.2.3 the protecting layers on the bridge deck as well as the railings will provide for a larger damping of the bridge which is also shown in this report. By doing tests with and without asphalt layers they concluded that the damping ratio, $\xi$, could differentiate with up to 3 percent units. When testing for different temperatures between -10°C and +40°C it was also shown that the temperature range would also give up to 3 percent units difference in $\xi$. Meaning that there could be large fluctuations in $\xi$ if there is a asphalt layer on top of the bridge deck.

In another article, Schubert et al. (2010) did a FE model in Abaqus of a simply supported SLT bridge to see:

- The influence of asphalt on the bridges damping ratio.
- The influence of asphalt on the bridges resonance frequency in the first and the second mode.
- If surrounding temperature impacts the asphalt.

Their program have a lot of similarities to the script that is created in this thesis and highlight a lot of important parameters which makes it relevant for this study. The study validates that the critical damping ratio increases quite a lot if the asphalt layer is included in the simulation compared to if the deck is modelled as only a timber slab. For the simply supported SLT deck the difference was 45 percent units in this particular case. If the asphalt is taken into account in the design process of the bridge there are several parameters that will be affected. Schubert et al. (2010) stated that both the frequency and the damping will change if there is a change in the outer temperature. This since the asphalt will change stiffness and material properties as the temperature varies. An advantageous correlation could be distinguished in damping for high temperatures, while for the frequency an advantageous correlation could be seen for low temperatures. The authors did however not study what these correlations might mean for the design of the bridge.

## 3.4 Design values and load models

As mentioned before, there is no exact design path for dynamic response in bridges since it is mainly a comfort criteria. Therefore, when designing a pedestrian bridge it becomes relevant what type of load model that is applied as well as other design values. These values and load models will determine how good the theoretical dynamic performance of the bridge will be and the aim is of course to be as accurate as possible. Jutila (1996) present a comparison between different load models that are used in the nordic countries showing that as the span increases, from 10 m and above, EC will allow for a much lower load intensity than what the national standards allow for in the nordic countries. The

author made the conclusion that over-design often was due to uncertainties. For example, to not take into account if the bridge is located in a crowded area or not. This leading to all bridges being designed for very crowded areas, consequently many bridges are over designed.
This is regarded in Appendix 2 of Sétra (Boniface et al., 2006) where modelling of the pedestrian load will mainly depend on 2 things: the load type that one pedestrian create and how crowded it can be on the bridge.

## 3.5 The dynamic effect in bridges from a changed span-to-depth ratio

A problem when constructing SLT decks is the lamination slips that might occur since the laminations are stressed together and are solely depending on friction, see Figure 2.1. In Table 5.1 in SS-EN 1995-2:2004, section 5, reduction factors can be found for stiffness parameters in laminated deck plates that are supposed to take this into account. However, Ekholm & Kliger (2014) wrote an article where they studied, inter alia, interlaminar slip and how it should be handled in design. In conclusion they found that a span-to-depth ratio of 29 and below will give a risk of interlaminar slip since the deck will be more stiff. In Euler-Bernoulli beam theory it is stated that the frequency is determined by the stiffness of the bridge - provided that the bridge is assumed to act as a beam - which makes this ratio an interesting parameter even for dynamic analysis.

In a study made by Shuk & Poon (2009), the span-to-depth ratio was studied in concrete bridges to see how it affects the structural behaviour. This study was specially performed to see if there is a difference now that bridges are build more slender and material efficient. They found that a serviceable span-to-depth ratio was between 20-40 depending on what type of concrete bridge it was and that the optimal values often occur between 22-38. Even though this study was made on concrete bridges it is confirmed by the Canadian Wood Truss Association (2017) that an appropriate span-to-depth ratio for timber decks are 25-35. In a paper written by Crocetti (2016), he explains that if timber bridges have a span that is longer than recommended it will give problems regarding buckling and deflection. He explains that timber in general has a very high structural efficiency when it comes to long spans regarding ULS.

## 3.6 Summary and key findings of literature review

From this literature review some key findings will be brought in to the selection of influencing parameters that will be studied and also considered in the evaluation:

- Sétra has a great correspondence to reality, which supports that the model made in this report is done according to Sétra.
- EC is a highly conservative method. Mostly due to adjustment factors and specified strength values. Therefore, it is essential to study how the dynamic calculations differ between the codes. Also, what is the reason to why EC is conservative?
- An hypothesis to why EC and dynamic design over all is perhaps more conservative than necessary is brought up by Jutila in Chapter 3.4. He means that there are

uncertainties which is probably due to the fact that there are no enforced codes, just design propositions that in the end is determined by the client. A discussion needs to be made if the dynamic design should be a part of the required design process.
- More studies are required to be able to determine more accurate eigenfrequencies etc. of bridges. This in order to reduce the magnitude of the safety margins.
- Damping and eigenfrequency changes due to coating has a large impact on dynamic behaviour. Therefore it would be of interest to study what happens when these parameters changes.
- Temperature has a large impact on damping and eigenfrequency. This needs to be studied further to see the extent of this impact.
- A stiff bridge is not always the best bridge. There has to be a junction between the dynamic design and the static design since a stiff SLT bridge might give problems in interlaminar slip, as Ekholm & Kliger showed in their study.
- An increase in critical damping ratio can be made with enough safety margin as shown in the test presented by Talja and Lilja.
- A span-to-depth ratio below 29 is critical for interlaminar slip. Therefore, it might be unnecessary to study dynamic response in bridges with ratios below this.

# 4

# Dynamic calculations according to different standards and codes

In this chapter the studied standards and codes are presented together with the relevant calculation steps. It presents thoroughly which checks need to be made, what limit values that are used and also in some extent why they perform the design process as they do. The study that was made in this thesis followed the limit values according to EC, however with the calculation steps of Sétra since Sétra has a great correspondence to reality, as stated in Chapter 3.6.

## 4.1 Eurocode 5 with the Swedish National Annex

The Eurocodes are standards that are implemented in the majority of the European countries and, in addition, are adopted in other parts of the worlds.

The Eurocode 5 contains information about design of timber structures. Part 1-1 (SS-EN 1995-1-1:2004) contains general rules and rules for buildings whilst part 2 (SS-EN 1995-2:2004) contains information about the design of timber bridges. The subsequent information in this chapter is taken from part 2. In EC 5 the dynamic calculations are included in the SLS design chapter.

During the time this master thesis was conducted a revision of EC 5 was in development where changes will be made in the dynamic design. Both with regards to design values and with making it a bit more elaborated(European Comisson, 2018). This could implicate that the dynamic design process is now considered to be of greater importance than before or that new significant knowledge hade been found.

One general design value that is used for all EC is the critical damping ratio. The recommended value, provided no other values have been stated, are shown in Table 4.1.

**Table 4.1:** Critical damping ratio for timber according to Eurocode 5

| | **Damping ratio: $\xi$** |
|---|---|
| **Without mechanical joints** | 0.01 |
| **With mechanical joints** | 0.015 |

There are different formulas for the calculation of vertical acceleration, according to Swedish standard SS-EN 1995-2:2004 (Annex B) in EC. It depends on how many persons that are crossing the bridge. The different directions are explained in Figure 2.2. The limit values for acceleration are $\alpha_{vert} < 0.7\ m/s^2$ and $\alpha_{hor} < 0.2\ m/s^2$, the calculating are made according to Equation (4.1) to (4.6).

For one person crossing, the vertical acceleration [m/$s^2$] is:

$$for\ \ f_{vert} \leq 2.5Hz: \quad \alpha_{vert,1} = \frac{200}{M \cdot \xi} \tag{4.1}$$

$$for\ \ 2.5Hz < f_{vert} \leq 5.0Hz: \quad \alpha_{vert,1} = \frac{100}{M \cdot \xi} \tag{4.2}$$

For one person running, the vertical acceleration [m/$s^2$] is:

$$for\ \ 2.5Hz < f_{hor} \leq 3.5Hz: \quad \alpha_{vert,1} = \frac{600}{M \cdot \xi} \tag{4.3}$$

For one person crossing, the horizontal acceleration [m/$s^2$] is:

$$for\ \ 0.5Hz < f_{hor} \leq 2.5Hz: \quad \alpha_{vert,1} = \frac{50}{M \cdot \xi} \tag{4.4}$$

For several persons crossing, the vertical acceleration [m/$s^2$] is:

$$\alpha_{vert,n} = 0.23 \cdot \alpha_{vert,1} \cdot n \cdot k_{vert} \tag{4.5}$$

For several persons crossing, the horizontal acceleration [m/$s^2$] is:

$$\alpha_{hor,n} = 0.18 \cdot \alpha_{hor,1} \cdot n \cdot k_{hor} \tag{4.6}$$

Where:

- M is the total mass of the bridge in kg
- $f_{vert}$ is the fundamental natural frequency i.e. eigenfrequency
- n is the number of pedestrians
- k is a coefficient from Figure B.1 in Annex B

## 4.2 Sétra

Sétra is a department within road and bridge construction and traffic safety in France. Sétra has published the technical guide: Assessment of vibrational behaviour of footbridges under pedestrian loading, written by Boniface et al. (2006). All information under Chapter 4.2 is according to Boniface et al., all tables and figure are either remade or made to summerize information from Sétra.

Experimental measurements have been performed which has shown that several parameters influence walking frequency, the number of steps per second, such as ground toughness, physiological characteristics etc. The measurements have resulted in one frequency range for walking and one for running.These frequency ranges are presented in 4.2 and the damping ratios that are used in Sétra are presented in 4.3.

**Table 4.2:** Frequencies for walking and running according to Sétra

| Designation: | Vibrations | Frequency [Hz] |
|---|---|---|
| Walking | Continuous | 1.6 – 2.4 (often taken as average of 2 Hz) |
| Running | Discontinuous | 2 – 3.5 |

**Table 4.3:** Critical damping ratio for timber according to Sétra

| | Minimum value | Average value |
|---|---|---|
| **Timber** | 1.5% | 3.0% |

In Sétra the dynamic analysis is made in six various steps explained below.

**Step 1: Determination of footbridge class**

Determined with regard to how much traffic the bridge should withstand according to Table 4.4. Class 4 does not require any dynamic analysis but if the bridge is very light it is recommended to calculate according to class 3.

**Table 4.4:** Footbridge class according to Sétra

| | |
|---|---|
| **Class 1** | Footbridges used by dense crowds or high pedestrian density areas |
| **Class 2** | Footbridges connecting popular areas |
| **Class 3** | Standard use |
| **Class 4** | Rarely used footbridges |

**Step 2: Determine comfort level**

The comfort level is decided by the owner of the bridge and can differ between four levels according to Table 4.5.

**Table 4.5:** Comfort level according to Sétra

| | |
|---|---|
| **Level 1** | Good comfort, barely any accelerations are perceived by the pedestrian |
| **Level 2** | Moderate comfort, accelerations can be perceived by the pedestrian |
| **Level 3** | Low comfort, accelerations can be perceived by the pedestrian |
| **Level 4** | Unacceptable accelerations |

The comfort level is achieved through calculations on accelerations that are undergone by the structure. These calculations are depending on different dynamic load cases. The acceptable accelerations are summarized in Table 4.6.

**Table 4.6:** Acceleration range for vertical and horizontal direction [m/$s^2$] according to Sétra

| | Level 1 | Level 2 | Level 3 | Level 4 |
|---|---|---|---|---|
| **Vertical vibrations** | 0 - 0.5 | 0.5 - 1 | 1 - 2.5 | 2.5 < |
| **Horizontal vibrations** | 0 - 0.1 | 0.15 - 0.3 | 0.3 - 0.8 | 0.8 < |

**Step 3: Determination of frequencies and if its necessary to perform dynamic load case calculations**

It is necessary to determine the natural vibrational frequency of the bridge in vertical and horizontal (transverse and longitudinal) direction. They are determined for two cases, empty bridge and loaded through bearing area ($700N/m^2$). The dynamic load case calculations are depending on the footbridge class and in which range its eigenfrequencies are. In Table 4.7 the different frequency ranges with corresponding risk of resonance are summarized.

**Table 4.7:** Frequency range for vertical and horizontal vibrations [Hz] according to Sétra

| **Vibrations** | **Range 1** | **Range 2** | **Range 3** | **Range 4** |
|---|---|---|---|---|
| *Risk for resonance* | *Max* | *Medium* | *Low* | *Negligible* |
| Vertical and Longitudinal | 1.7 - 2.1 | 1.0 -1.7 and 2.1 -2.6 | 2.6 - 5.0 | <1.0 and >5.0 |
| Transverse | 0.5 - 1.1 | 0.3 -0.5 and 1.1 -1.3 | 1.3 - 2.5 | <0.3 and >2.5 |

Depending on footbridge class, Table 4.4, and frequency range, Table 4.7, different load cases are applied according to Table 4.8. If the table results in "non", no calculations are required.

**Table 4.8:** Correlation between footbridge class and frequency range to get a load case

| | **Frequency Range** | | |
|---|---|---|---|
| | 1 | 2 | 3 |
| **Footbridge class 1** | Case 1 | Non | Non |
| **Footbridge class 2** | Case 1 | Case 1 | Case 3 |
| **Footbridge class 3** | Case 2 | Case 2 | Case 3 |

**Step 4: Calculations with dynamic load case (if needed)**

There are tree different cases of dynamic load. The load must be placed on the most dangerous pattern, case 1 to 3 gives load per $m^2$. The reduction factor $\psi$ can be retrieved from Figure 4.1 and density $d$ of the pedestrian crowd is according to Table 4.9. $n$ is depending on d as the area of the bridge deck times $d$.

**Case 1:** Spread to dense crowds

Dynamical load $[m^2]$ according to each direction:

$$Vertical_1(v) = d \cdot (280N) \cdot cos(2\pi f_v t) \cdot 10.8 \cdot (\xi/n)^{1/2} \cdot \psi \tag{4.7}$$

$$Transverse_1(t) = d \cdot (35N) \cdot cos(2\pi f_t t) \cdot 10.8 \cdot (\xi/n)^{1/2} \cdot \psi \tag{4.8}$$

$$Longitudinal_1(l) = d \cdot (140N) \cdot cos(2\pi f_l t) \cdot 10.8 \cdot (\xi/n)^{1/2} \cdot \psi \tag{4.9}$$

**Case 2:** Very dense crowds

Dynamical load $[m^2]$ according to each direction:

$$Vertical_2(v) = d \cdot (280N) \cdot cos(2\pi f_v t) \cdot 1.85 \cdot (1/n)^{1/2} \cdot \psi \qquad (4.10)$$
$$Transverse_2(t) = d \cdot (35N) \cdot cos(2\pi f_t t) \cdot 1.85 \cdot (1/n)^{1/2} \cdot \psi \qquad (4.11)$$
$$Longitudinal_2(l) = d \cdot (140N) \cdot cos(2\pi f_l t) \cdot 1.85 \cdot (1/n)^{1/2} \cdot \psi \qquad (4.12)$$

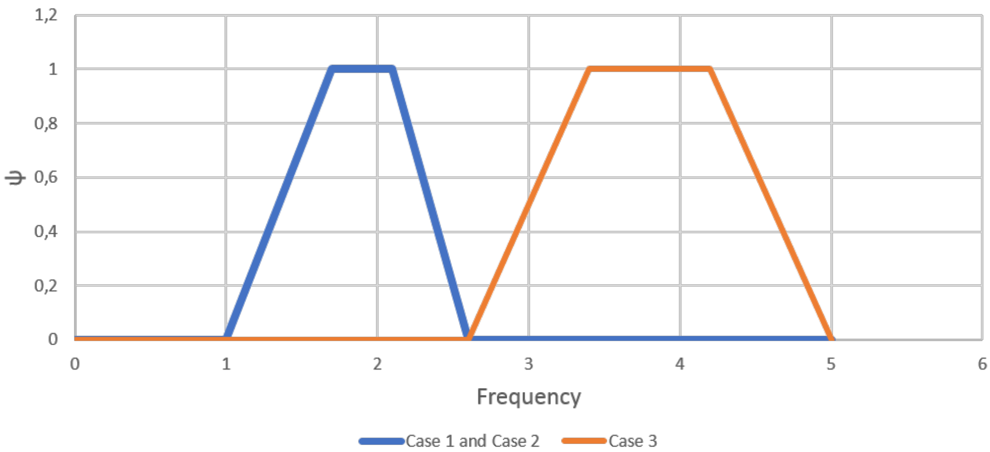


**Figure 4.1:** Reduction factor $\psi$ in case of walking for vertical vibrations

**Table 4.9:** d is the density of the pedestrian crowd and is chosen based on footbridge class.

| Class | Density d of the crowd |
|---|---|
| **1** | 1.0 |
| **2** | 0.8 |
| **3** | 0.5 |

**Case 3:** Effects of the second harmonic of crowds

In dynamic load case 3, the effect of second harmonic caused by pedestrians at double the frequency of the first harmonic is evaluated. It should be taken into account in class 1 and 2, see Table 4.4. Reductions for load case 3 for class 1 and 2 according to Table 4.10.

**Table 4.10:** Reduction of effects of the second harmonic of crowds according to Sétra

| **Direction** | **Reduction** |
|---|---|
| **Vertical** | 280 N reduced to 70 N |
| **Longitudinal** | 140 N reduced to 35 N |
| **Transverse** | 35 N reduced to 7 N |

**Step 5: Modification of the footbridge**

If the demands are not fulfilled, the project needs to be restarted with changing parameters affecting the dynamic response, until calculations show acceptable values.

## 4.3 AASHTO

In the US, AASHTO is the official set of standards that are used for construction all over the country. The name stands for American Association of State Highway and Transportation Officials but despite the name, the standard is used for all type of constructions that involve infrastructure of any kind. AASHTO is not a part of or a product of the government but it is nevertheless a non-profit association that has no connection to politics which makes it considered as the official guideline to follow. (American Association of State Highway and Transportation Officials, 2014)

AASHTO LRFD (load and resistance factor design) guide is specialised for the design of pedestrian bridges and was published by AASHTO in December 2009. The following information is all taken from AASHTO LRFD. In the guide it is described that the pedestrian load should be applied in order to enhance the deformation pattern, to get the worst case scenario. The load of pedestrians standing on the bridge should be taken as 90 psf which is a slightly lower value than the one EC uses which is converted to 105 psf. However, the load factor in EC is 1.5 while the load factor in Aashto is 1.75 which gives the resulting end value of 158 psf for both cases.

The limit for eigenfrequency values are set to 3 Hz for the vertical vibration modes when live load is not considered. If the eigenfrecuency is above 3 Hz and if the second harmonic response is not an issue in the bridge, no further investigation needs to be made. However, if a dynamic investigation is required it has to include four parts:

- Which frequency and load magnitude the pedestrians provide
- The lock-in effect (see Chapter 2.3.3), the effect of phase shift between the pedestrians
- Structural damping effects
- Acceleration limits

In order to investigate if the bridge has sufficient dynamic response, the following demands has to be met:

$$f_{vert} \geq 2.86 \cdot ln\frac{180}{W} \tag{4.13}$$

$$W \geq 180 \cdot e^{-0.35 \cdot f_{vert}} \tag{4.14}$$

Here $W$ is the dead load of the structure.

## 4.4 CHBDC

In Canada, The Canadian Highway Bridge Design Code (CHBDC) is used for design of bridges. It is an extensive code that covers all important aspects of bridge construction. The CHBDC is the single code that is used and it is applied in all provinces of Canada.

Since the code is very thorough, a number of subcommittees have been involved in respective chapters. This, in order to have experts work only on their area of expertise to make the code as accurate and safe as possible. The code is also revised continuously to be adapted to current construction prerequisites.

Section nine in CHBDC is specified to wood structures, however, in Section 3.4.4 of CHBDC it is specified how to handle dynamic loads in SLS. Every chapter starts with an introduction to why the check needs to be made. For example, the "Pedestrian Bridges" chapter is introduced by explaining why SLS can be determinig in some bridge cases whilst ULS is determining for other bridges. For example, the swaying effect that was observed in the Millennium bridge in London and the problem that occurred is described. See chapter 2.2 for further explanation.

In CHBDC it is written that the reason why design of pedestrian bridges often will be determined by SLS is that the critical damping might be below 1%. This makes vibration a significant aspect since the vibration can be excited if it is not dampened. As shown in Figure 2.3.

It is stated that the maximum frequency a pedestrians can produce ($f_{ped}$) while walking is 3 Hz and upper limit for joggers is 4 Hz. However, these are rare cases and the step frecuency is rarely over 2.5 Hz. A frequency analysis should be made of the bridge to retrieve eigenfrequencies and if there are any frequencies below 4 Hz, a special analysis should be made to see if the bridge meet the acceleration demands.

If a model of the structure is created for dynamic analysis, the code states that it is accurate to neglect any torsional effects for pedestrian bridges. Exceptions are made for wider bridges since the supports then will be too weak and rotation might occur. However, this will not be the case in the standard pedestrian bridge.

It is stated in the code that it cannot be assumed that the first eigenfrequency of a bridge will provide the worst case acceleration. Nevertheless, a correlation has been made that for a $n$-span bridge, the worst case will be found in the $n$ first modes.

**Design method:**
Multiple studies indicate that several persons walking will not walk in phase with each other, as explained further in Chapter 2.3.3. On the contrary they will dampen each others steps. Therefore, the load used in CHBD is for only one person of 70 kg walking on the bridge since that will provide for the worst case scenario. For the bridge to pass the initial check, this person should not give the bridge an acceleration greater than the limit values provided in Figure 4.2 which is remade from the original code. These limit values are compiled from tests on vibrations in both floors and bridges since it has been shown that there is a connection in perceived discomfort. For this acceleration analysis they state that all torsional effects can be over looked when it comes to continuous bridge decks which further validate the limitation made in this thesis.

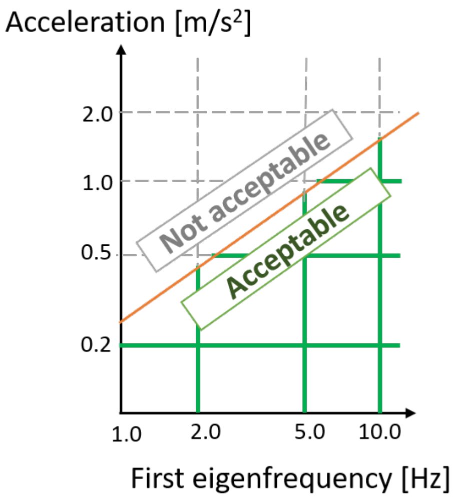


**Figure 4.2:** CHBDC: Acceleration criteria for a single pedestrian of 700 N

The acceleration can be calculated in two different ways, either by doing a full dynamic analysis or by a simplified method. The simplified method can be used for continuous bridges with maximum three spans that together acts as a beam.

If a full dynamic analysis is performed, the step frequency of the pedestrian should be set to to:

$$f_{ped}=\min\begin{cases} f_1 \\ 4Hz \end{cases}$$

and the speed that the pedestrian walks in should be set to:

$$V_{ped}=\min\begin{cases} 0.9 \cdot f_1 \\ 2.5m/s \end{cases}$$

The footfall force $Force_{ped}$ is then calculated according to the following equation and is applied with the tempo that the pedestrian is walking in:

$$Force_{ped} = 180 \cdot sin(2 \cdot \pi \cdot f_{ped} \cdot t) \tag{4.15}$$

If the simplified approach is used, the acceleration is calculation according to the following equation:

$$a = 4 \cdot \pi \cdot f_1^2 \cdot W_s \cdot K \cdot \chi \tag{4.16}$$

Where $W_s$ is static deflection if the bridge is loaded with a point load of 70 kg, $K$ is a span-to-span factor and $\chi$ is a factor that is a function of damping in the bridge. $K$ and $\chi$ can be gathered from tables in the code.

# 5

# Modeling

The purpose of this chapter is to thoroughly explain how the Python script was established and how one can apply it both in the industry but also for further studies in this area. The chapter includes a background to the software that are used together with a guide to how the script has been developed in order to give more accurate results.

With this, one should be able to use this chapter if they want to redo the simulations or continue with this study.

## 5.1 Used software

When establishing the script, three software was used: A FEM software called Brigade/-PLUS (Abaqus), a programming language called Python and a spreadsheet software called Excel. All software is linked to each other and together they can perform a dynamic analysis of an SLT bridge with input data chosen by the one performing the study, see Figure 5.1. Below is a short description to how and why the different software is used.


**Figure 5.1:** Description of how software is linked to each other

### 5.1.1 Excel: input and output file

Initially, data is entered in the Excel input file. For instance material parameters, geometry values, boundary conditions etc. The input data from the Excel file will be loaded into the python script which will make is possible to model any SLT bridge within limitations. Input can be changed in the yellow and green boxes while the white boxes are predefined values or values that change in connection to other values. See Figure 5.2 for an overview of the Excel file.

| | | Geomteri | | Stödförhållanden | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | Längd [m] | | U1 | U2 | U3 | R1 | R2 | R3 |
| Antal span | 3 | Yttre stödlinje | 0,4 | Stöd 1 | 1 | 1 | 1 | 1 | 0 | 0 |
| | | Spann 1 | 12 | Stöd 2 | 0 | 1 | 1 | 1 | 0 | 0 |
| | | Spann 2 | 16 | Stöd 3 | 0 | 1 | 1 | 1 | 0 | 0 |
| | | Spann 3 | 12 | Stöd 4 | 0 | 1 | 1 | 1 | 0 | 0 |
| | | Spann 4 | | Stöd 5 | | | | | | |
| | | Spann 5 | | Stöd 6 | | | | | | |
| | | Yttre stödlinje | 0,4 | | | | | | | |
| Klimatklass | 2 | | | | | | | | | |
| | | Bredd platta | 4 | | | | | | | |
| | | Höjd platta | 0,53 | | | | | | | |
| | | Höjd beläggning | 0,085 | | | | | | | |
| | | Total längd bro | 40,8 | (ändras automatiskt) | | | | | | |

| Indata för dynamiska beräkningar | | |
|---|---|---|
| GC-Klass | 2 | |
| *Folkmassans täthet* | 0,8 | (ändras automatsikt) |
| Maxvärde Egenfrekvens | 10 | (Enligt EC) |
| Beläggning densitet | 195,5 | [kg/m^2] |
| Densitet räcke | 85 | [kg/m^2] |
| Högd räcke | 1,5 | [m] |
| Mesh storlek | 0,29 | |

**Figure 5.2:** Overview of some parts of the Excel input file.

The results from the dynamic analysis are printed in a new Excel file, the so-called output file, which the user can use for the post-processing.

The main purpose of both Excel files is to create an easier way to use the script if one is not familiar with Python. In the finished program one only has to do changes in the Excel input file and then run the program, Brigade/PLUS, in order for the results to be shown in the Excel output file.

### 5.1.2 Python: the script

Python is a programming language that has an constructs- and object oriented approach which helps programmers to write clear and logical scripts. A strong advantage with Python is that it works great with many other software and is therefore a good language to combine with Brigade/PLUS.

After the input data has been entered in Excel, the Python script reads the input and the information is sent to Brigade/PLUS by pressing "run script". The script will then automatically generate the geometry of a bridge model, run the analyses, apply the loads etc. Since everything occurs automatically, the script provides for a much simpler design process for the user that just has to enter the input data and then interpret the result.

### 5.1.3 Brigade/PLUS

Brigade/PLUS is a structural analysis application which is an expansion package of the FEM software Abaqus. It has the primary function of designing bridges and one can get multiple useful outputs such as stresses, strains, section forces, accelerations etc.

The main contribution that Brigade brings to Abaqus is the load models that corresponds to different vehicles. You can choose to download different codes to the work directory based on which country the bridge will be built in that provides for the correct load model and apply this to the bridge.

At COWI, there is a inquiry for a parametrizised model that calculates the dynamic response for pedestrian bridges in the software Brigade/PLUS since it would be more time efficient. However, Brigade/PLUS itself can only design according to EC and currently EC is very conservative, as shown in Chapter 4, and therefore not beneficial to use. By running the script that is implemented with Sétras design values and load models it is possible to use Brigade/PLUS for the dynamic design.

## 5.2 Establishing the model

The Python script itself is built up by sections which are further explained under each sub chapter. The script has two separate ways of applying load. The first, more simplified version, is called Model 1 and the second, more complex version, is called Model 2. Model 1 has limitations while Model 2 has been developed beyond these limitations. A consequence to the development in application of load is that it takes longer time to run the simulations. A more thorough description of the scripts is found under Subchapter 5.2.4 and 5.2.5.

### 5.2.1 Geometry and Boundary conditions

In Section 1 of the script, the geometry together with material properties are established with the connection to Excel. The properties of the SLT deck are chosen according to Eurocode. Fixed values that have been used is: $E_{0,mean}$ = 13 000 MPa, $E_{90,mean}$ = 300 MPa, $G_{0,mean}$ = 650 MPa and $G_{90,mean}$= 65.0 MPa. All mass are added as nonstructural mass.

After the geometry of the bridge is set, the bridge deck is divided into parts. One part per span, see Figure 5.3. The bridge deck is thereafter divided into different sets, either into subsets which is made in Model 1 or in nodesets as done in Model 2, see Subchapter 5.2.4 and 5.2.5.

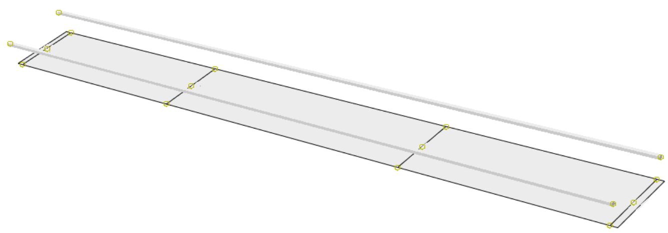

**Figure 5.3:** Model: Different divisions of bridge deck.

The boundary conditions (BC) are set according to what is chosen in the Excel input file. For this study, as mentioned in Chapter 1.2, all supports are fixed for vertical movement and horizontal movement in the transverse direction of the bridge (it is simply supported) but this can be changed if the user wishes. The most left support, Support 1, is also locked for horizontal movement in the longitudinal direction of the bridge. The BCs were modelled as line supports, as can be seen later in in Figure 6.1, which will make them stiff enough to handle twisting effects.

The mesh is chosen to be 0.29 m which is a sufficient size for pedestrian SLT bridges. This is the value that is used at COWi and the mesh is also verified in Chapter 5.3. See Figure 5.4 for the mesh when applied to an arbitrary SLT bridge.

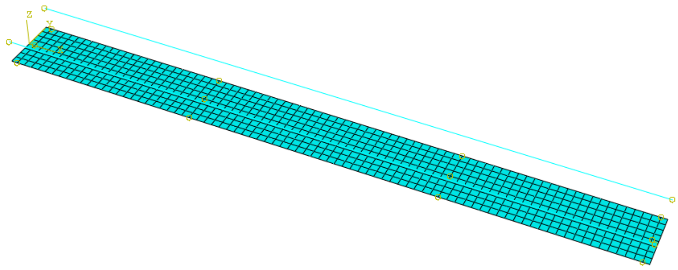

**Figure 5.4:** Model: Example of meshed bridge

### 5.2.2 Frequency analysis

In Section 2, a frequency analysis is made on the bridge to see what eigenfrequency or eigenfrequencies the bridge has as well as the corresponding oscillation modes. All eigenfrequencies below the limit value are extracted. This limit value is 5 Hz for design according to EC but it can be changed in the Excel input file. The frequency analysis is made for two different models, one without crowd load and one with crowd load, see Chapter 4.2 for further information. The study is made only without crowd load since the study only consider correlation and not absolute values of the acceleration. An output ODB file is created with the eigenfrequencies. These are used in the following acceleration study, in order to see which eigenfrequency that will give the highest acceleration.

### 5.2.3 Establishing and applying the loads

In Section 3 the live load in the form of a vertical force is applied on each mode below the eigenfrequency limit. If there are no eigenfrequencies below the limit value, an error message appears. The load is placed in the worst case scenario. This means that if the deformation is upwards, the load will be an upwards force. The deformation will therefore guide the script to which direction the load should be applied. The loads are calculated according to Sétra, see Chapter 4.2.

### 5.2.4 Model 1: Evenly distributed load

To apply loads onto different parts of the bridge deck a division of the surface is done. Each span is divided in the middle, see yellow dotted line in Figure 5.5. This entails six surfaces, so called load sets, where the load can be applied vertically, either in positive or negative direction.

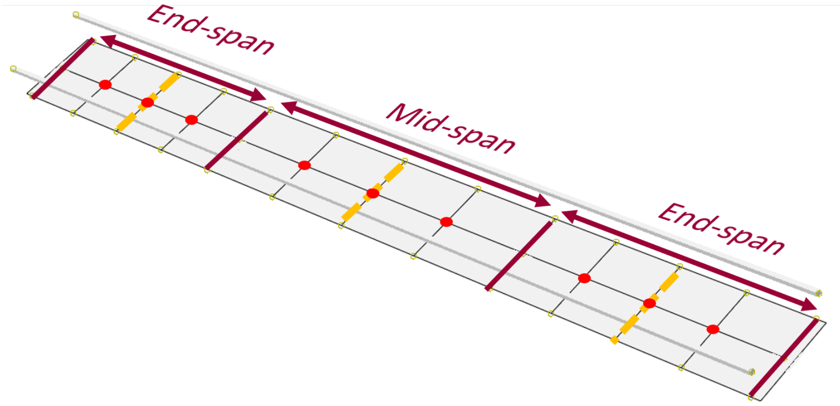


**Figure 5.5:** Model 1: Each span partitioned in the middle is shown together with the nodes used for deformation check

In Figure 5.5, nine dots are visualized, there are three evenly distributed dots within each span, so-called nodes. The nodes are used for the deformation check that will determine which direction the loads will have. See the example in Figure 5.6.

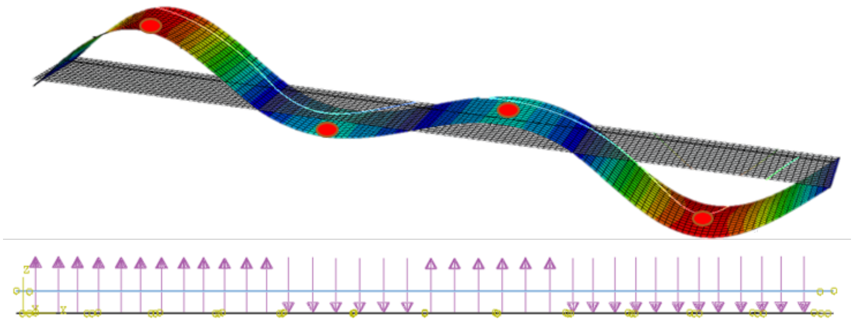

**Figure 5.6:** Model 1: Evenly distributed load within each load set, visualized for mode 2

One limitation with Model 1 is that it is only accurate when end spans are of equal length and the mid span is equal or longer than the end spans. The reason for this limitation is that when the deformation change does not take place in the center of a span, which it tends not to do for long end span, the script will not detect this.

Another limitation of Model 1 is that it is only accurate for vertical modes, consequently not for transverse or twisting modes. For the study in this thesis it is not a problem since the SLT pedestrian bridges are treated as plates and are assumed to be stiff enough in the transverse plane. Thus, no horizontal modes will occur. This supposition can also be supported by the Canadian code, CHBDC. Twisting however can happen even though it is a rare phenomenon in pedestrian bridges since they are rarely wide enough.

Even though Model 1 is limited it has the benefit of running quickly and is therefore advantageous to use for geometries within the limitations. In the study made in this thesis, Model 1 is used for all studies within its limitation due to the faster run time.

### 5.2.5 Model 2: Multiple point loads

Model 2 is designed as a development of Model 1, with the aim to exclude the limitations. Overall, the establishment of Model 2 is the same as for Model 1 and the only difference is the application of load. Instead of applying an evenly distributed load onto a surface, numerous point loads are created and applied, one for each node. The bridge deck consists of multiple nodes and the amount of nodes are depending on mesh size. For Model 2 to work, the mesh size have to be small enough for the point loads to create the same behavior as for a distributed load. This was checked in a verification.

The nodes are divided into end nodes and mid nodes, see Figure 5.7. The load calculated according to Sétra are in $N/m^2$ and therefore has to be recalculated to $N$ per node. The point load is calculated separately for end nodes and for mid nodes because of the difference in area, see edge area and mid area in Figure 5.7.

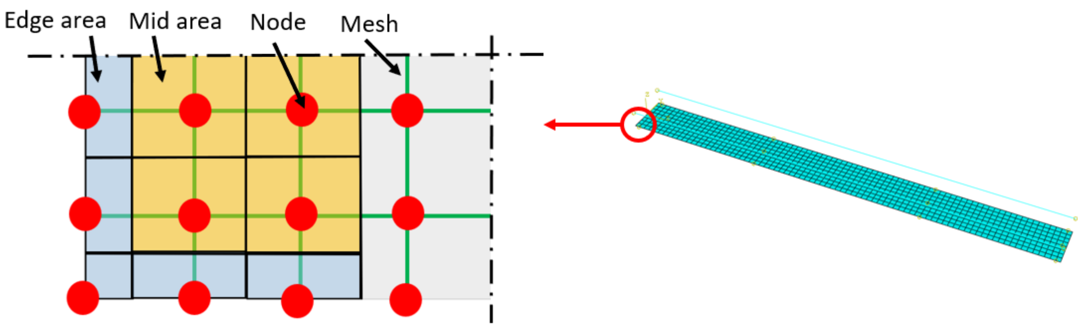


**Figure 5.7:** Model 2: End part of a bridge deck with load distribution for mid and edge nodes

Deformation is calculated for each node and the load is applied depending on positive or negative deformation, same as for Model 1, see Figure 5.8.

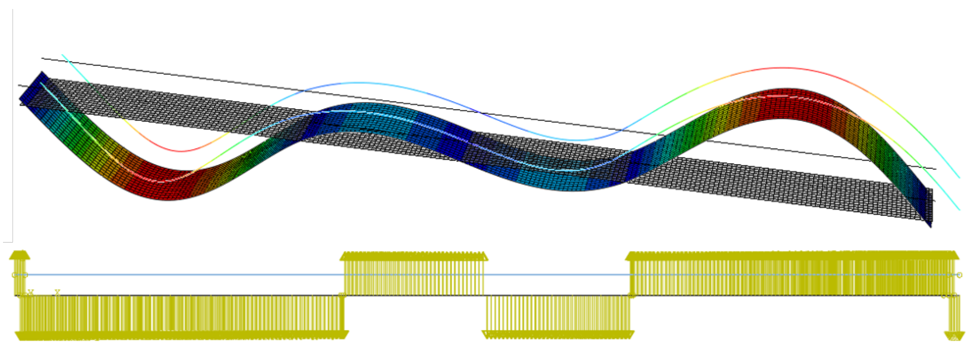

**Figure 5.8:** Model 2: Point load at every node, visualized for mode 2

Model 2 can handle rotational modes and since all nodes are run separately the applied load can adapt to any mode appearance. One disadvantage is that the script takes much longer to run than Model 1. In this study, Model 2 is only used for studies when end-spans are longer than the mid-span due to the run time.

### 5.2.6 Acceleration analysis

In section 4, a steady state modal analysis is made to find the maximum acceleration. See 2.4 for a more thorough explanation of how this is done. The max acceleration is first found for each node, then the maximum value out of the nodes accelerations is picked and transferred to the Excel output file. The script finds the maximum acceleration and it also locates the corresponding eigenfrequency together with the other eigenfrequencies below 5 Hz.

## 5.3 Verification of Python script: eigenfrequency comparison

To verify that the script is correct it is compared with the dynamic analysis made at COWI for three pedestrian bridges in the software Nastran. The script is verified for both load applications, which will be called Model 1 and Model 2. Notice that there are not any difference in the script except for how the load is applied. The bridges are located in three different locations in Sweden: Hasselfors, Härlövsängaleden and Vångavägen. In this chapter the comparisons are presented with the margin of error presented in Table 5.2, 5.3 and 5.5. All the existing bridges are presented in Table 5.1. They all have a mid span that is longer than the end-spans which means that the limitations for Model 1 are fulfilled and consequently, both Model 1 and Model 2 should receive the same eigenfrequencies. However, the acceleration values might differ due to the different application of load. The mode shape looks the same for all bridges, they only differ in dimensions such as length. Figure 5.9 visualizes the different mode shapes in pictures taken from Brigade/PLUS model.

As can be seen in the figures and tables below, the verification of the models was successful. The deviations from the Nastran values amounted to maximum 2.3% which can be considered neglectable since the margin of error for a FEM analysis is larger than that. This could be shown in the study presented by A.Talja and H.Lilja where the deviation was up to 15% due to fluctuations in material properties.

What can also be seen is that the deviation is the same for Model 1 and Model 2 which was expected since the script is the same except for how the loads are applied.

**Table 5.1:** Dimensions of existing bridges

| | Span 1 | Span 2 | Span 3 | Width | Thickness | Coating |
|---|---|---|---|---|---|---|
| **Vångavägen** | 13 m | 17 m | 13 m | 4 m | 0.630 m | 85 mm |
| **Hasselfors** | 12 m | 16 m | 12 m | 4 m | 0.630 m | 85 mm |
| **Härlövsängaleden** | 19 m | 21.5 m | 19 m | 3 m | 0.64 m | 85 mm |

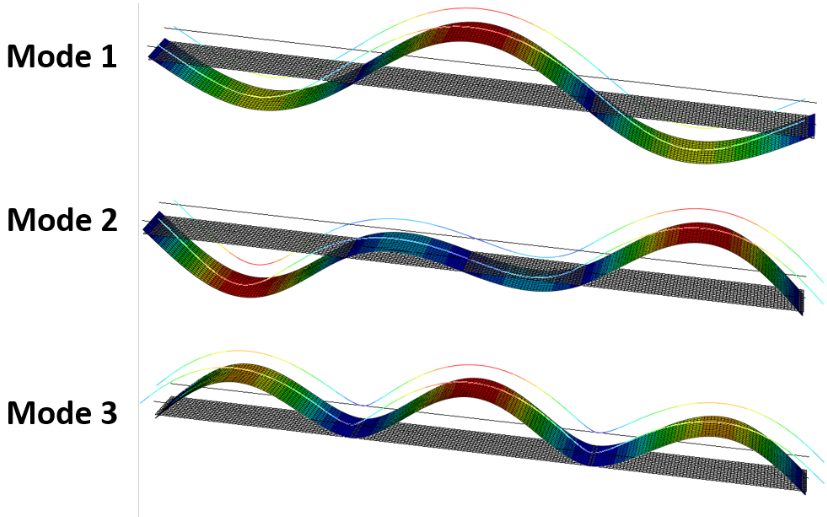


**Figure 5.9:** Mode 1, 2 and 3 visualized for a three span bridge.

**Table 5.2:** Verification of frequency against Hasselfors bridge

| Model | Mode 1 [Hz] | Mode 2 [Hz] | Mode 3 [Hz] |
|---|---|---|---|
| Without crowd load | | | |
| Nastran | 5.41 | No value | No value |
| Model 1 | 5.43 | - | - |
| Model 2 | 5.43 | - | - |
| Deviation | 0.36% | - | - |
| With crowd load | | | |
| Nastran | 5.26 | No value | No value |
| Model 1 | 5.27 | - | - |
| Model 2 | 5.27 | - | - |
| Deviation | 0.19% | - | - |

No verification on acceleration can be made for Hasselfors bridge since no eigenfrequencies are below 5 Hz.

**Table 5.3:** Verification of frequency against Härlövsängaleden bridge

| Model | Mode 1 [Hz] | Mode 2 [Hz] | Mode 3 [Hz] |
|---|---|---|---|
| Without crowd load | | | |
| Nastran | 2.68 | 3.69 | 4.75 |
| Model 1 | 2.73 | 3.77 | 4.86 |
| Model 2 | 2.73 | 3.77 | 4.86 |
| Deviation | 1.8% | 2.1% | 2.3% |
| With crowd load | | | |
| Nastran | 2.60 | 3.58 | 4.62 |
| Model 1 | 2.65 | 3.66 | 4.72 |
| Model 2 | 2.65 | 3.66 | 4.72 |
| Deviation | 1.9% | 2.2% | 2.1% |

**Table 5.4:** Verification of acceleration against Härlövsängaleden bridge, without crowd load

| Model | Acceleration $[m/s^2]$ |
|---|---|
| Nastran | 0.489 |
| Model 1 | 0.483 |
| Model 2 | 0.483 |
| Deviation | 1,2% |

**Table 5.5:** Verification of frequency against Vångavägen bridge

| Model | Mode 1 [Hz] | Mode 2 [Hz] | Mode 3 [Hz] |
|---|---|---|---|
| Without crowd load | | | |
| Nastran | 4.62 | No value | No value |
| Model 1 | 4.61 | - | - |
| Model 2 | 4.61 | - | - |
| Deviation | 0.21% | - | - |
| With crowd load | | | |
| Nastran | 4.49 | No value | No value |
| Model 1 | 4.49 | - | - |
| Model 2 | 4.49 | - | - |
| Deviation | 0% | - | - |

**Table 5.6:** Verification of acceleration against Vångavägen bridge, without crowd

| Model | Acceleration $m/s^2$ |
|---|---|
| Nastran | 0.264 |
| Model 1 | 0.259 |
| Model 2 | 0.259 |
| Deviation | 1.9% |

# 6
# Parameter study

In this chapter each performed study is presented in three sections: The procedure of the study in the form of why and how, the results from the study followed by an evaluation of the results. The procedure section contains a presentation of the parameter that are studied together with a explanatory table and a figure which can be used to interpret the result. Note that the evaluation is only for that specific study and that a compiled evaluation of the whole thesis can be found in the following discussion chapter.

The studies were performed for multiple bridges with different geometry ratios. This was to show that the trend of the curves are the same for several bridges which consequently means that the forthcoming evaluation do not need to consider the variation. The different parameters are referred to as different studies and the variation in geometry is referred to as different cases, as can be seen in Table 6.1.

**Table 6.1:** Performed parameter studies

| **Study** | **Studied** | **Case** |
|---|---|---|
| A | Span-to-span ratio | A1 to A11 |
| B | Span-to-depth ratio | B1 to B4 |
| C | Positioning of supports | C1 to C11 |
| D | Critical damping ratio | D1 to D6 |
| E | Density | E1 to E6 |

All studies with end-span $\leq$ mid-span are performed with load application according to Model 1. The rest are performed according to Model 2. To easier detect what or which parameters that have been changed in the respective study, these have been highlighted in each table as a yellow column.

In Figure 6.1 an explanation to what the geometrical terms are referring to is shown. In Appendix A the complete result for all variations in geometry are presented.

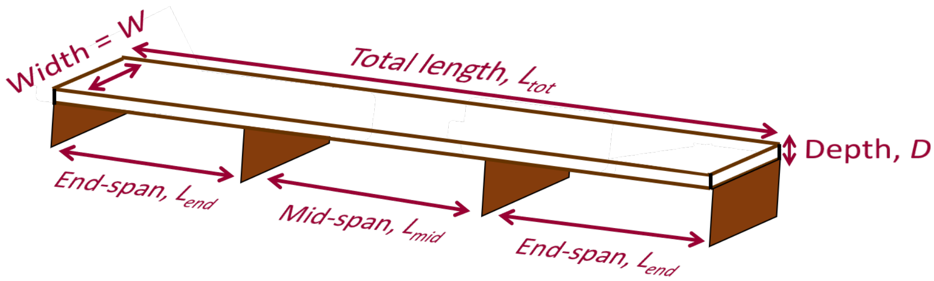


**Figure 6.1:** Geometrical terms visualized on a bridge

The output values are the maximum acceleration and its corresponding eigenfrequency. However, in the evaluation the resulting accelerations are not considered with regards to pass or fail since the absolute value is not relevant but rather the correlation of the curve.

## 6.1 Study A: Span-to-span ratio

### 6.1.1 Procedure

The first studied parameter was the span-to-span ratio in relation to the maximum acceleration. As mentioned, the length of a bridge is determined by the site conditions and cannot be altered. This study is therefore purely academic, in order to see if there is a maximum length a bridge can have before the dynamic response is too severe.

In order to study this specific parameter, other parameters need to be fixed. In Study A, the mid span was set as a fix value and the end spans were set to be equally long. The ratio between mid and end spans were varied with 41 values between 0.6 to 1.4, see Figure 6.2. These 41 values are later referred to as $x$ in Table 6.2.

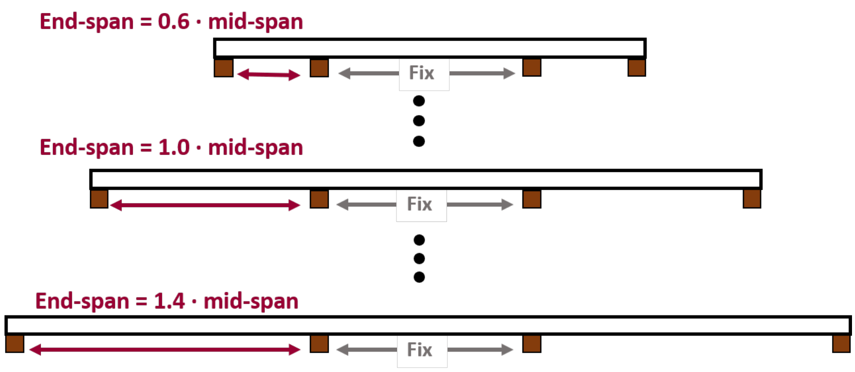


**Figure 6.2:** Study A explained where three of the 41 points with varying end-spans are visualized. 0.6, 1 and 1.4. Later referred to as x.

Study A was executed for eleven different cases. The values of the governing mid span was chosen with regards to information from Chapter 2.3.2 where it is specified that the ratio between the width and the span needs to be above four, leading to the following minimum span:

$$\frac{Span}{Width} > 4 \quad and \quad width = 4m \quad \rightarrow \quad 4 \cdot 4m (= 16m) < Span \tag{6.1}$$

Therefore 16 meter is the lower limit for the longest span. Consequently, the study starts at 16 m as is presented in Table 6.2. The ratio between mid span and thickness of the bridge deck was set to a value between 25 to 35, the reason for the chosen ratios are described in Study B below. Since the depth is a fixed value, the spans will be varied and that will provide for the following values of the mid span:

**Table 6.2:** Studied values for span-to-span ratio where $x$ is 41 values in range $0.6 \leq x \leq 1.4$

| Case | $L_{mid}/D$ [-] | $\xi$ [%] | $\rho$ [kg/m$^3$] | Geometry $L_{tot}$ [m] | $L_{mid}$ [m] | $L_{end}$ [m] | D [m] | W [m] |
|---|---|---|---|---|---|---|---|---|
| A1 | 25 | 1.0 | 600 | Varies | 16.00 | $x \cdot L_{mid}$ | 0.64 | 4 |
| A2 | 26 | 1.0 | 600 | Varies | 16.64 | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| A3 | 27 | 1.0 | 600 | Varies | 17.28 | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| A4 | 28 | 1.0 | 600 | Varies | 17.92 | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| A5 | 29 | 1.0 | 600 | Varies | 18.56 | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| A6 | 30 | 1.0 | 600 | Varies | 19.20 | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| A7 | 31 | 1.0 | 600 | Varies | 19.84 | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| A8 | 32 | 1.0 | 600 | Varies | 20.48 | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| A9 | 33 | 1.0 | 600 | Varies | 21.12 | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| A10 | 34 | 1.0 | 600 | Varies | 21.76 | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| A11 | 35 | 1.0 | 600 | Varies | 22.40 | $x \cdot L_{mid}$ | 0.64 | 4.0 |

### 6.1.2 Results

The result for four selected bridges are presented in Figure 6.3, the result for all eleven cases can be found in Appendix A.

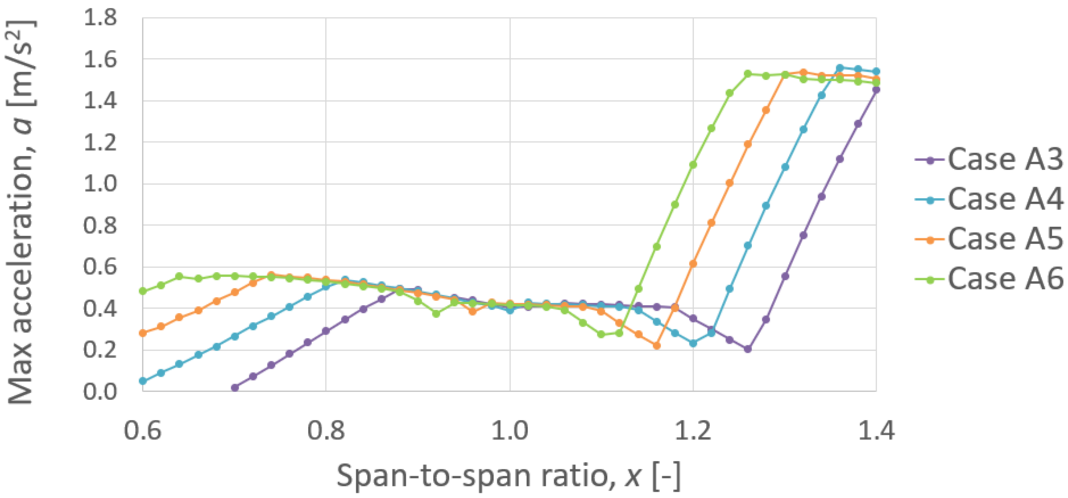


**Figure 6.3:** Result for Study A, cases A3 to A6

### 6.1.3 Evaluation

The result from Study A shows a trend where the acceleration stabilizes when the ratio between the spans are almost equal. When the end span becomes longer than the mid span there is a critical point where the acceleration increases rapidly. The trends are the same for the different cases and one can see that the absolute values for the accelerations are very similar regardless of the case all though with a shift in the response. This might signify that the total length of the bridge and the span-to-depth ratio is of less importance compared to the ratio between the spans. This also indicates that longer bridges can be constructed but then it is more important that the ratio between the spans is one or less. In conclusion, attempting to change the span ratios will be of higher importance as the total length of the bridge increases.

Since the result is affected by the total length, Study C was performed with a fixed total length and varying support positioning to see more thoroughly how the ratios affect the accelerations.

Another reflection that is important to keep in mind is that despite of this result, the bridge cannot have infinite slenderness. It is therefore important to connect the dynamic analysis with the static to see if it is strong enough.

## 6.2 Study B: Span-to-depth ratio

### 6.2.1 Procedure

The second parameter that was studied was the maximum acceleration in relation to the span-to-depth ratio. Currently when you design a bridge, if it is not approved from a dynamic perspective, the solution is to thicken the plate. It is therefore already established that an increased deck depth will have a positive effect on the dynamics. This is because the eigenfrequencies will be higher and therefore be less critical.

Study B was performed in order to see how much the bridges dynamic behaviour differ when the span-to-depth ratio increases and if the current solution of thickening the plate is the most optimal one.

Each test was performed with a fixed depth and a varying mid span length that corresponded to 41 points in the interval from 25 to 35, see Figure 6.4.

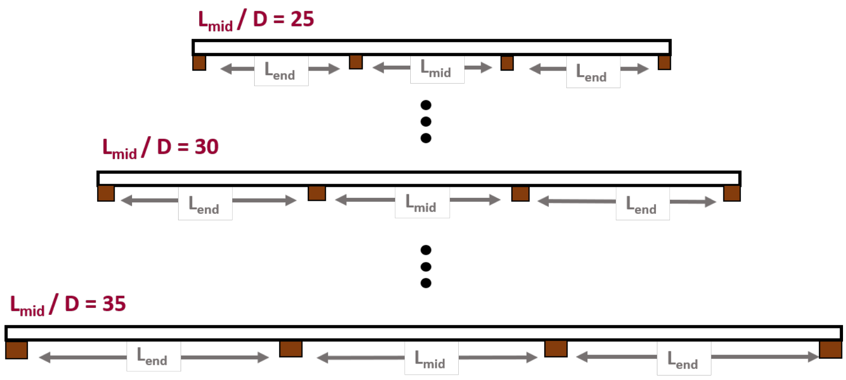


**Figure 6.4:** Study B, case B3, explained where three of the 41 tests with varying ratios are visualized

The study treated four different cases with varying span-to-span ratios, see Table 6.3.

**Table 6.3:** Studied values for Span-to-depth ratio where $x$ is 41 values in range 25 $\leq$x$\leq$ 35

| Case | $L_{mid}/D$ | $\xi$ | $\rho$ | Geometry | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | $L_{tot}$ | $L_{mid}$ | $L_{end}$ | D | W |
| | [-] | [%] | [kg/m$^3$] | [m] | [m] | [m] | [m] | [m] |
| B1 | 25 $\leq$x$\leq$ 35 | 1.0 | 600 | Varies | 16.0 $\leq$x$\leq$ 22.4 | 0.6 $\cdot$ $L_{mid}$ | 0.64 | 4.0 |
| B2 | 25 $\leq$x$\leq$ 35 | 1.0 | 600 | Varies | 16.0 $\leq$x$\leq$ 22.4 | 0.8 $\cdot$ $L_{mid}$ | 0.64 | 4.0 |
| B3 | 25 $\leq$x$\leq$ 35 | 1.0 | 600 | Varies | 16.0 $\leq$x$\leq$ 22.4 | 1.0 $\cdot$ $L_{mid}$ | 0.64 | 4.0 |
| B4 | 25 $\leq$x$\leq$ 35 | 1.0 | 600 | Varies | 16.0 $\leq$x$\leq$ 22.4 | 1.2 $\cdot$ $L_{mid}$ | 0.64 | 4.0 |

## 6.2.2 Results

The result for all four cases are presented in Figure 6.5 with corresponding explanation in Table 6.3.

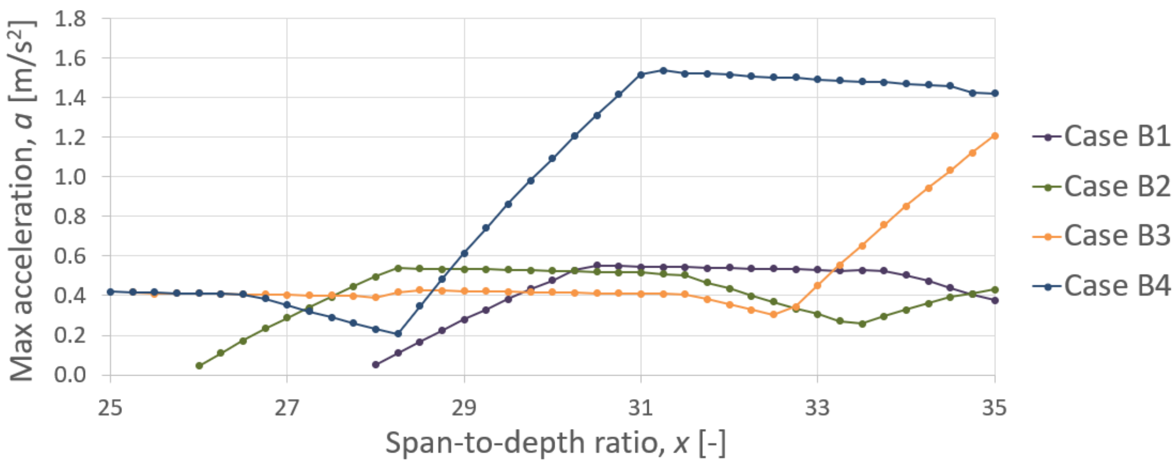


**Figure 6.5:** Result for Study B

## 6.2.3 Evaluation

It is hard to draw any conclusions from this study since there is no obvious correlation. What can be seen from the graphs is that the span-to-depth ratio does not appear to have a large influence for ratios below 31-33. There is a slight enhancement of the acceleration as the ratio increases however, one can expect a larger increase. It is apparent that further evaluation need to be done on this parameter. Perhaps another way to study the parameter would be to change the depth instead of the span lengths. Yet another way to performe this study could be to have a fixes total length to remove the influence of total length on the acceleration. It is possible that if the study is performed with these parameters fixed, a more clear correlation could be distinguished.

Howver, when the depth is fixed, only the stiffness in longitudinal direction changes. If the depth had been changed instead of the length, the stiffness had changed both in longitudinal and transverse direction. Consequently, this might have given a different result.

As one can expect, the peak in acceleration comes earlier for Case B3 and B4 which is most probably due to the fact that the bridge is more slender.

As can be supported by the literature presented in Chapter 3.5, the ultimate span-to-depth ratio seem to be around 27-33 depending on the total length. However, what needs to be considered and what Ekholm & Kliger wrote in their paper, the ULS perspective cannot be neglected here since longer spans (and ultimately a higher slenderness) will provide for a larger deflection and less strength in the deck.

## 6.3 Study C: Positioning of supports

### 6.3.1 Procedure

The third and final geometry ratio study that was performed was how the positioning of supports affect the bridges behaviour. As earlier mentioned, the current solution for when a bridge has insufficient dynamic performance is to increase the depth of the deck, providing the bridge with more mass and stiffness. However, a more material efficient solution that would be more environmental friendly, would be if the positioning of the supports could provide for a different dynamic performance instead. A lot of geometrical parameters are predetermined but, as mentioned in Chapter 2.5, the positioning of supports might be changed partially. Therefore, Study C was performed to compare the acceleration with a change in support positioning. Here, the total length of the bridge was fixed and only the supports were moved in relation to each other, see Figure 6.6.

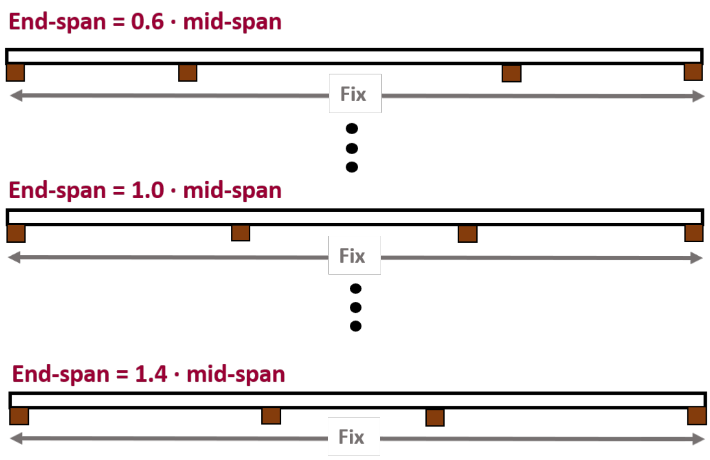


**Figure 6.6:** Study C explained where three of the 41 tests with varying span lengths conditions are visualized

The ratios between the end spans and mid span was changed with 41 different ratios where the end span length varied from $0.6 \cdot midspan$ to $1.4 \cdot midspan$, referred to as $x$ in the Table 6.4.

**Table 6.4:** Studied values for different positioning of supports with fixed span length where $x$ is 41 values in range 0.6 ≤x≤ 1.4

| Case | $L_{mid}/D$ [-] | $\xi$ [%] | $\rho$ [kg/m$^3$] | Geometry $L_{tot}$ [m] | $L_{mid}$ [m] | $L_{end}$ [m] | D [m] | W [m] |
|---|---|---|---|---|---|---|---|---|
| C1 | 25 | 1.0 | 600 | 48.00 | $L_{tot}/(2x+1)$ | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| C2 | 26 | 1.0 | 600 | 49.92 | $L_{tot}/(2x+1)$ | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| C3 | 27 | 1.0 | 600 | 51.84 | $L_{tot}/(2x+1)$ | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| C4 | 28 | 1.0 | 600 | 53.76 | $L_{tot}/(2x+1)$ | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| C5 | 29 | 1.0 | 600 | 55.68 | $L_{tot}/(2x+1)$ | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| C6 | 30 | 1.0 | 600 | 57.60 | $L_{tot}/(2x+1)$ | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| C7 | 31 | 1.0 | 600 | 59.52 | $L_{tot}/(2x+1)$ | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| C8 | 32 | 1.0 | 600 | 61.44 | $L_{tot}/(2x+1)$ | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| C9 | 33 | 1.0 | 600 | 63.36 | $L_{tot}/(2x+1)$ | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| C10 | 34 | 1.0 | 600 | 65.28 | $L_{tot}/(2x+1)$ | $x \cdot L_{mid}$ | 0.64 | 4.0 |
| C11 | 35 | 1.0 | 600 | 67.20 | $L_{tot}/(2x+1)$ | $x \cdot L_{mid}$ | 0.64 | 4.0 |

### 6.3.2 Results

The result for five selected bridges are presented in Figure 6.7 with corresponding explanation in Table 6.4. The result for all eleven cases can be found in Appendix A.

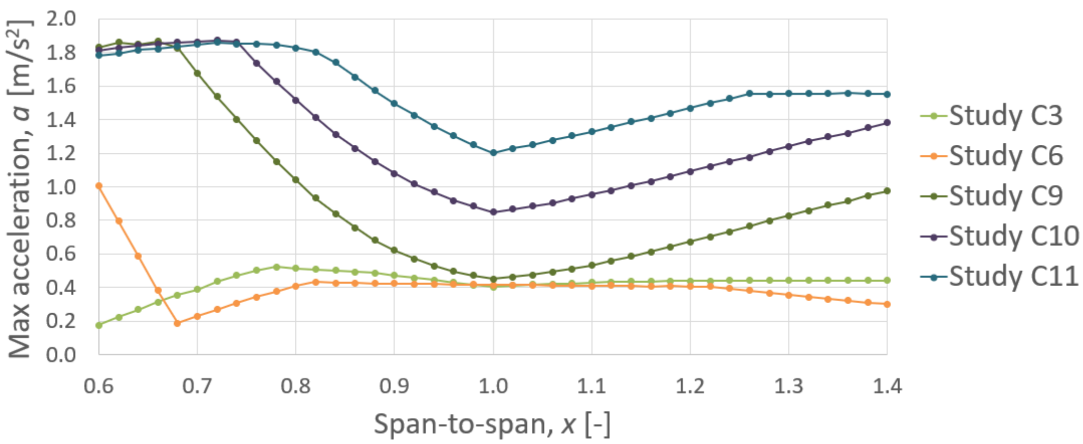


**Figure 6.7:** Result for Study C, five cases shown in the graph

### 6.3.3 Evaluation

One can see that for longer bridges (shown by Case 9-11) there is a distinct advantage with having a span-to-span ratio close to one, meaning that all spans have the same length. While, for the shorter bridges (shown by Case 3 and 6) the difference is basically negligible. The longer the bridge is, the more the accelerations will dive when the ratio is approaching one. This indicates that if you are designing a bridge and the acceleration response is too big, there can be an advantage to move the supports in order to make the ratio closer to one. This further validates the conclusion from Study A, where it is stated that the ratios will have a bigger influence for longer bridges.

What might be the basis of this behavior is the effect of stiffness. Both Study A and C show that there is a slight advantage in having stiffer end spans rather than a stiff mid span. This is however something that need to be further investigated from a structural mechanical perspective.

Another solution that this might lead to is the possibility to only strengthen one or two of the spans. Since the deck is constructed as a plate there might be a problem in only making a certain span thicker but you can however stiffen one specific part of the bridge. Perhaps with the help of the railings, with cross braces or by tightening the steel bars more at certain places. This will require more studies to see any other negative effect that might arise from making the bridge inhomogeneous in stiffness.

To further connect to Study A where it was found that the total length of the bridge may be of less importance than the span ratios. We could see in Study A that the bridges performed well as long as the ratios were one or less. What we can see here is that if a ratio above one does not provide for a longer total length (since the total length is fixed in Study C), the ratio above one will not be a problem either. Thus, the "problem" in Study A was not the long end spans but merely that the total length of the bridge became too long.

## 6.4 Study D: Critical damping ratio

### 6.4.1 Procedure

The critical damping ratio study was performed by comparing different critical damping ratios to the maximum acceleration. The span-to-span ratio was fixed to one which means that all spans have equal length, see Figure 6.8. The reason for this was to make sure that all modes have the same mode shape and consequently the result was easier to interpret due to less frequency jumps and more consistent results, see Chapter 6.6.

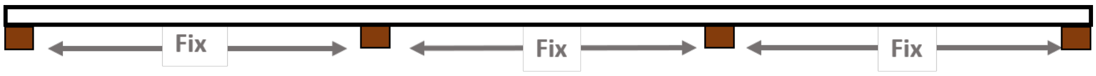


**Figure 6.8:** Study D explained with bridge deck geometry fixed while the critical damping is changed

The critical damping ratio was studied for values between 0.5% to 10% with 41 points in between. The study was performed for six different cases, see 6.5.

**Table 6.5:** Studied values for critical damping ratio where $x$ is 41 values in range $0.5\% \leq x \leq 10\%$

| | | | | Geometry | | | | |
|---|---|---|---|---|---|---|---|---|
| Case | $L_{mid}/D$ [-] | $\xi$ [%] | $\rho$ [kg/$m^3$] | $L_{tot}$ [m] | $L_{mid}$ [m] | $L_{end}$ [m] | D [m] | W [m] |
| D1 | 25 | $0.5 \leq x \leq 10$ | 600 | 48.00 | 16.00 | 16.0 | 0.64 | 4.0 |
| D2 | 27 | $0.5 \leq x \leq 10$ | 600 | 51.84 | 17.28 | 17.28 | 0.64 | 4.0 |
| D3 | 29 | $0.5 \leq x \leq 10$ | 600 | 55.68 | 18.56 | 18.56 | 0.64 | 4.0 |
| D4 | 31 | $0.5 \leq x \leq 10$ | 600 | 59.52 | 19.84 | 19.84 | 0.64 | 4.0 |
| D5 | 33 | $0.5 \leq x \leq 10$ | 600 | 63.36 | 21.12 | 21.12 | 0.64 | 4.0 |
| D6 | 35 | $0.5 \leq x \leq 10$ | 600 | 67.20 | 22.40 | 22.40 | 0.64 | 4.0 |

### 6.4.2 Results

The result for three selected bridges are presented in Figure 6.9 with corresponding explanation in Table 6.5. The result for all six cases can be found in Appendix A.

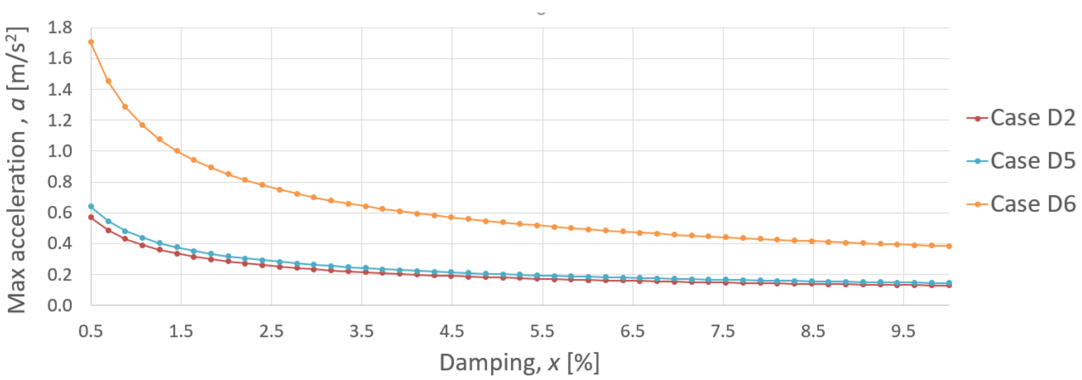


**Figure 6.9:** Result for study D, Case D2, D5 and D6

### 6.4.3 Evaluation

The effect of damping in a structure is a discussion with a lot of different statements. The Canadian code argue that the critical damping could possibly be below 1% in pedestrian bridges due to their low mass whilst a lot of other studies and tests show that the damping often is bigger than the 1% value that is used in design. Which the testing performed on Älvsbackabron by Jansson & Svensson for instance showed. The fixed value for damping that is used in design is based on the material properties of timber and does not include the effects that protective layers, railings or other decoration might have. By including the damping properties from these additional masses, the damping has been shown to be up to 10%. A problem with the critical damping is however that it cannot be calculated in advance and must be measured on the actual structure which makes it necessary to set the value lower than actually anticipated. In the study presented by A.Talja and H.Lilja it was shown that the a damping ratio can be increased to at least 2%, possibly 3% without obtaining any negative consequences. This result is something that can be supported by several other studies. Additional support of the fact that an increase of the damping ratio to at least 2% would not make the bridge unsafe is the fact that Sétra are using standard damping ratios between 1.5-3%.

Based on the result from these previous studies together with the tendency that our result shows, an increase to 2% critical damping ratio will not make the designs unsafe but rather be a step in making the calculation process less conservative. As you can see from our results in Figure 6.9, for certain bridges a difference from 1% to 2% will make the bridges approved for construction instead of requiring a resizing, as for the example in Case D6. As the correlation is negatively exponential, the difference from 1% to 2% will give a massive difference in maximal acceleration and a further increase in damping ratio will have less influence on the maximum acceleration. Hence, we can see no major reason to increase the damping ratio further if that will give a too big uncertainty.

One interesting aspect of the critical damping value is that it is a fixed value. In codes, there is a difference in damping based on materials and when the type of joints are changed. However, as stated previously in this report the damping is a complex parameter which is affected by a lot of things. To use the same value for basically all bridges is unreasonable and more influencing parameters should be added when selecting the damping ratio. For example if the bridge has a protecting layer such as asphalt, what boundary conditions the bridge has etc. This is something that will require further studies before it can be implemented.

A counterargument to an increase of critical damping ratio and an important thing to consider is that as the slenderness and total mass of bridges gets smaller and smaller the damping will also be reduced since it is dependent on mass. Currently, a fixed critical damping value is used in Eurocode (and in several outher standards) as presented in Chapter 2.2.3. This value might have to be reconsidered if the mass of structures are remarkably reduced.

# 6.5 Study E: Density

## 6.5.1 Procedure

The density study was performed by comparing different densities to the maximum acceleration. The study was performed in the same way as for study D with all spans of equal length, see Figure 6.10. The only difference being that damping is fixed to 1% and the density varies between 370 and 630 with 41 points in between. The study was performed for six different cases with a varying span-to-depth ratio, see Table 6.6. Note that a lower mean density means lower stiffness.

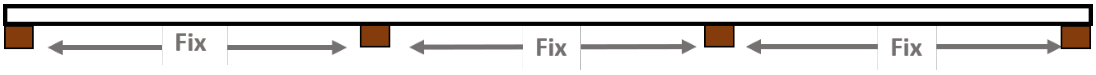


**Figure 6.10:** Study D explained with bridge deck geometry fixed while the density is changed

**Table 6.6:** Studied values for density variation where $x$ is 41 values in range 370 $\leq$x$\leq$ 630

| Case | $L_{mid}/D$ [-] | $\xi$ [%] | $\rho$ [kg/$m^3$] | Geometry $L_{tot}$ [m] | $L_{mid}$ [m] | $L_{end}$ [m] | D [m] | W [m] |
|---|---|---|---|---|---|---|---|---|
| E1 | 25 | 1.0 | $370 \leq x \leq 630$ | 48.00 | 16.00 | 16.0 | 0.64 | 4.0 |
| E2 | 27 | 1.0 | $370 \leq x \leq 630$ | 51.84 | 17.28 | 17.28 | 0.64 | 4.0 |
| E3 | 29 | 1.0 | $370 \leq x \leq 630$ | 55.68 | 18.56 | 18.56 | 0.64 | 4.0 |
| E4 | 31 | 1.0 | $370 \leq x \leq 630$ | 59.52 | 19.84 | 19.84 | 0.64 | 4.0 |
| E5 | 33 | 1.0 | $370 \leq x \leq 630$ | 63.36 | 21.12 | 21.12 | 0.64 | 4.0 |
| E6 | 35 | 1.0 | $370 \leq x \leq 630$ | 67.20 | 22.4 | 22.4 | 0.64 | 4.0 |

## 6.5.2 Results

The result for four selected bridges are presented in Figure 6.11 with corresponding explanation in Table 6.6. The result for all six cases can be found in Appendix A.

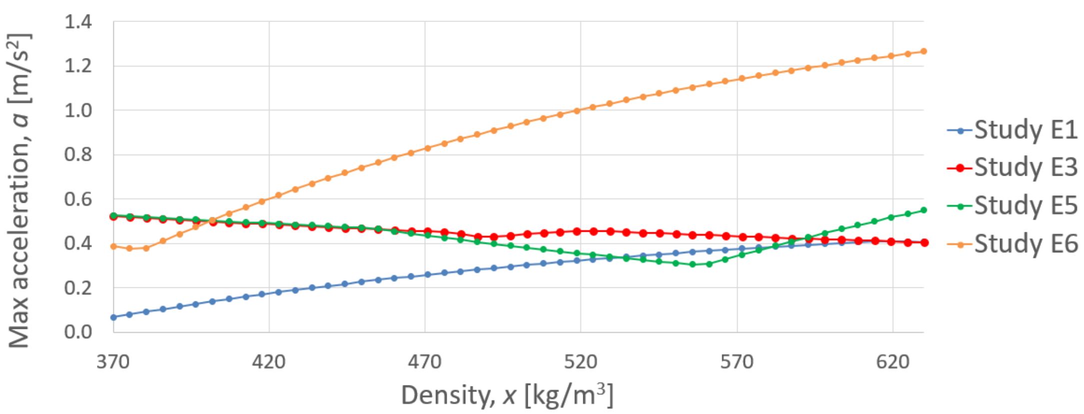


**Figure 6.11:** Result for study E, Case E1, E3 and E6

### 6.5.3 Evaluation

As seen in Figure 6.11, no distinguishing correlation can be seen between the different graphs. We know from Equation 2.1 that an increase of the linear density will give a smaller frequency since the density does not affect the stiffness. In turn, the frequency has an effect on the acceleration but this will be dependent on the range that the eigenfrequency is located in. One can see both an increase and a decrease in acceleration as more density is added. This makes the result ambiguous and thus hard to draw any conclusion.

However, this shows that even though an excessive value of the density is used to be on the safe side for ULS calculations it is not obvious that it is on the safe side with regards to SLS response. The correlation might be both positive or negative from a dynamic perspective. Currently the values for the density are exaggerated from a "safe side" perspective (for the static analysis) but this case would imply that the exaggeration of density could be negative from a dynamic perspective. As mentioned it is a well known fact that a lower mean density means lower stiffness and since the stiffness affects the dynamic response we know that it will be of some importance. Consequently, further studies need to be done to map how the dynamics behaviour is affected by the density and as mentioned before a better correspondence between the static and the dynamic design needs to be implemented in the codes.

## 6.6 Frequencies for different span-to-span

A check for frequencies in the range 0-7 Hz was made with the aim to describe and fortify the behaviour of the frequencies since an inequality was observed in some of the studies. The inequality was observed in the form of "jumps" or drastic changes in direction of the acceleration. This can be observed in Figure 6.2, 6.5 and 6.7.

The results of the frequency check are presented in Figure 6.12. The mid-span was arbitrarily chosen to be 19 m and the span-to-span ratio varied between 0.6 to 1.4 times the mid span since those are the ratios that were used in the parameter study.

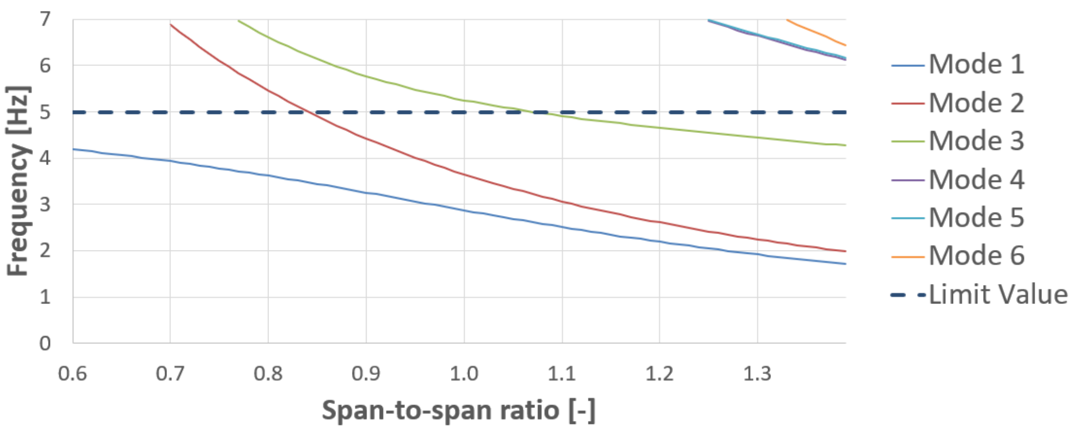


**Figure 6.12:** Behaviour of frequencies as the span-to-span ratio increases, mode 1, 2 and 3 visualized in Figure 1.1

As can be seen in this grapgh, as the ratio increases there will appear more eigenfrequencies below the limit value of 5 Hz since the total length of the bridge will be longer thus making it more slender. Different frequency ranges will provide for different loads which can cause the highest acceleration value to jump. In Chapter 2.2.2 it is mentioned that the highest load does not imply the worst case scenario and additionally it is mentioned in CHBDC that the first mode does not imply the worst case scenario. Therefore, if there are multiple eigenfrequencies below 5 Hz, any of them might give the maximum acceleration which can cause the jump. The jump is this due to of the maximum acceleration changes its corresponding eigenfrequency. To validate the results from the studies in this thesis and to show that they are reasonable, the frequency jumps are plotted more thoroughly for one bridge with a frequency jump from Studies A, B and C. In Figure 6.13 the frequency analysis is presented for Case A6, in Figure 6.14 the frequency analysis is presented for Case B4 and in Figure 6.15 the frequency analysis is presented for Case C6.

From Figure 6.13, 6.14 and 6.15 it is clear that the acceleration jumps when the corresponding frequency changes. To understand more thoroughly how the frequency span affect the resulting acceleration, see Chapter 4.2.

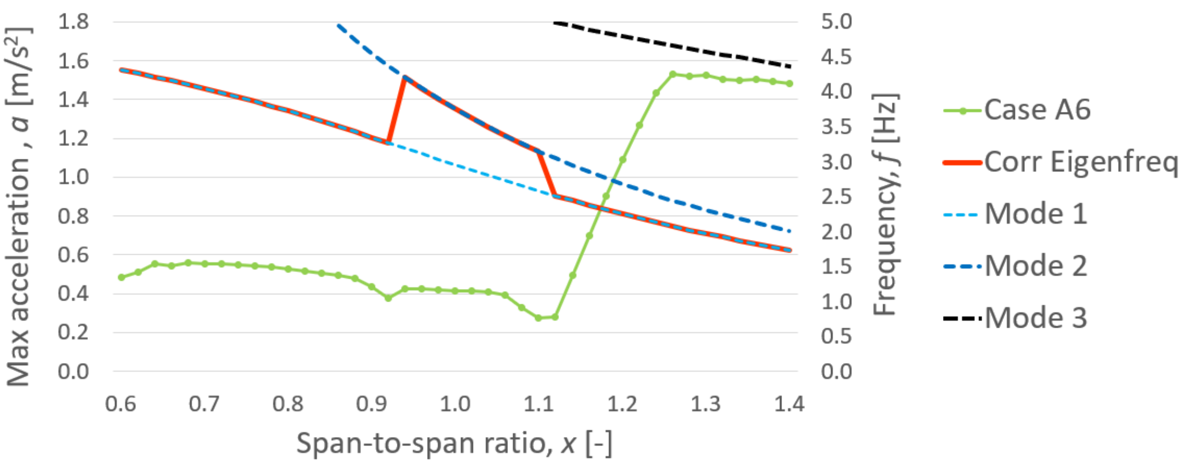


**Figure 6.13:** Case A6 with corresponding eigenfrequency and all modes beneath 5Hz

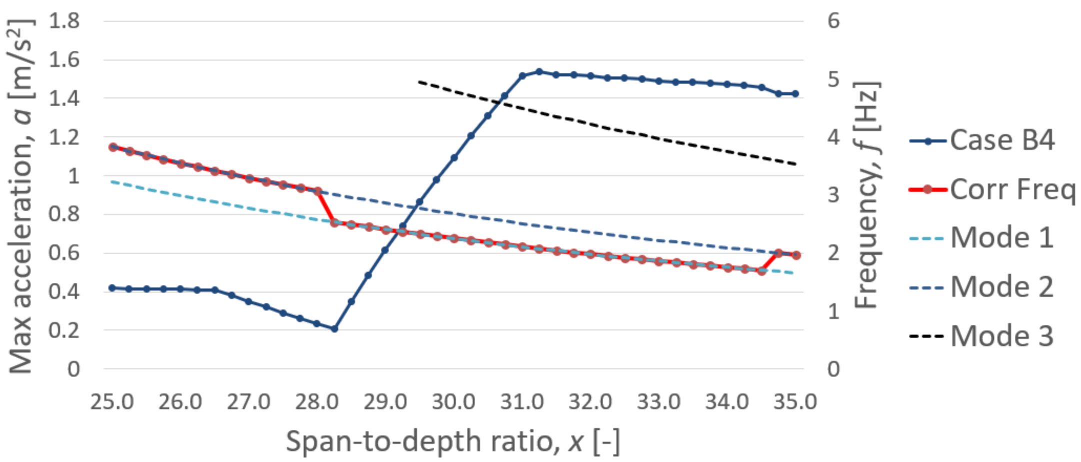


**Figure 6.14:** Case B4 with corresponding eigenfrequency and all modes beneath 5Hz

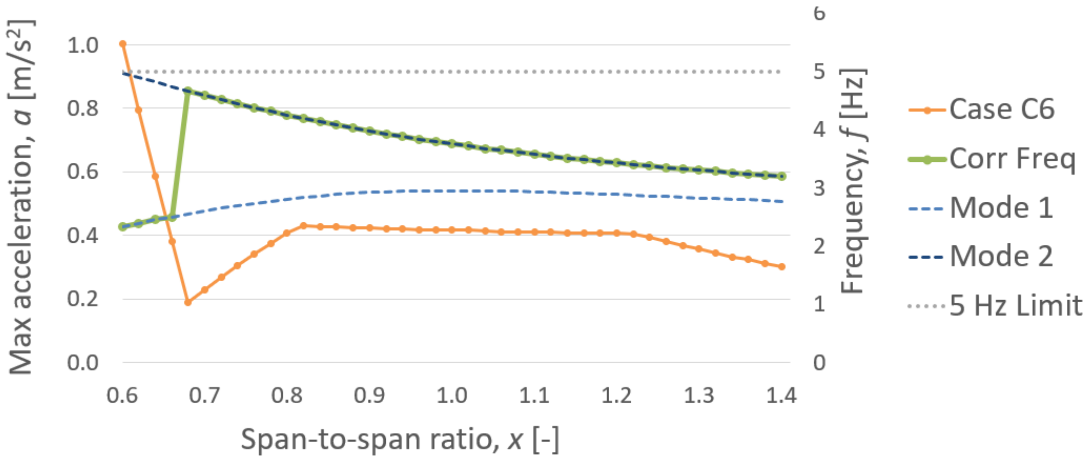


**Figure 6.15:** Case C6 with corresponding eigenfrequency and all modes beneath 5Hz

# 7
# Discussion

In the following chapter the report is tied together by a discussion connected to the key findings from the theoretical part of this report together with the results from the study. Several aspects has been treated and in the end of this chapter you will find suggestions for further aspects that could be investigated. These could be used for further development of this thesis but also as a separate study for a thesis with a different purpose.

## 7.1 Comparison between codes

With an increase of span lengths, slender designs and by using material more efficiently, the risk of dynamic problems increases. Therefore, it is important to design according to a code that provides for accurate calculations and limit values that reflects the reality as good as possible.

By comparing the studied codes and standards (AASHTO, CHBDC, Sétra and EC5 part 2) it is clear that EC has the highest limit for critical frequency of 5 Hz for pedestrian bridges. While for Sétra it is stated that the risk is very low between 2.6-5.0. For AASHTO the limit value is 3 Hz and for CHBDC it is 3 Hz or 4 if the bridge is to be used by a lot of runners. EC does provide simplified, and thus conservative, calculations if the frequency is below 5 Hz but rather recommends a magnification of the geometry should be made. This causes bridges that are designed according to EC to be of larger dimensions than necessary which is both material inefficient and uneconomical. This is also the reason why it is preferable to use Sétra to perform an acceleration analysis for the frequencies below the critical range.

An interesting aspect is that EC both have the highest limit value of the critical frequency and furthermore provides for very conservative calculations for accelerations. Making it basically impossible to design a bridge with eigenfrequencies below 5 Hz according to ECs calculations. An eigenfrequency below 5 Hz does not at all mean that the bridge will not perform good enough from a dynamic perspective as you can see both from the three bridges that were used in the verification but also from the arbitrary bridges that were studied. For EC to have both the highest limit value for critical frequency and not to provide a method for an acceleration analysis does not provide for a sustainable development.

Numerous results from studies performed in this report, mainly study A, B and C, shows that that SLT bridges with eigenfrequencies below 5 Hz, often with a good margin, can withstand the requirement of 0.7 m/$s^2$ (EC).
Therefore, we suggest that further inquiry of providing calculations if eigenfrequencies exists below 5 Hz should be done for all codes. Consequently, a check for accelerations,

similar to the one Sétra provides, should be implemented. This, in order to get a better utilization rate of the material. The statement that an eigenfrequency below 5 Hz does not have to result in bad dynamic response is confirmed in both the American and the Canadian standard where the bridge are assumed to be sufficient as long as it has a eigenfrecuencies are above 3-4 Hz.

**The implementation of a dynamic design process in codes**
Codes are developed continuously, some codes more often than others. This in order to keep the design process up-to-date and to be able to apply for more modern constructions. In a society that is growing at the same time as the environment gets drained of its resources it is important to utilize all aspects of a construction.

As mentioned previously, the construction industry has improved their structures a lot over the past years by making them more slender and material efficient. However, there are still a lot of things in the design process that are very conservative. As it has been shown in previous events such as the Tacoma bridge and the Millenium bridge, the dynamic design might be of highest interest. A discussion should be made regarding the implementation of a more thorough dynamic design for light-weight pedestrian bridges. Currently a lot of focus is on the static design of a bridge which is reasonable for bridges that will carry heavier loads. However, for pedestrian bridges there should be a parallel design process for the static and the dynamic part. A counterarguments on this might be that it will be very complicated and might take a lot of time to make these designs to coincide. But nowadays, the majority of bridge designs are performed using some sort of FEM analysis software which can be developed to calculate dynamic response as well, as made in this thesis.

There are a lot of parameters to consider when designing bridges, specially when taking both statics and dynamics into account. There are specialists with different areas of expertise and in CHBDC they let each specialist compile a chapter that reflects their respective area. Perhaps, a global code should be made that is composed by specialists in the same way. This would be a way to make the design process easier and one can also in a less complicated way compare bridges in different countries.

## 7.2 Suggestions for further investigations

When producing the script, some simplifications and limitations were assumed. As the project proceeded the script was developed and these limitations were dissolved. Therefore, further development of the script is not necessary only minor changes if other parameters should be studied. However, the script can now be used to incorporate even more studies and additional tests. The following is a compilation of both suggested studies that has already been mentioned previously in this thesis as well as suggested studies that might be of interest but that need to be further investigated in theory first.

**Further investigations that can be made, based on theory:**

- The loss of stiffness in the deck due to loss of tension in steel bars. This is something that need to be more thoroughly investigated since this thesis only treats variations in geometrical stiffness and not material stiffness. Crocetti et al. states that if the reduction of stiffness is below 40% it is still good enough with regards to static design. He however does not mention the dynamic design. He also says that the anchorage system behaves differently for thin decks. For pedestrian bridges the deck is often more slender than for road bridges which could indicate that a more thorough study should be made for this.

- The effect of damping in the rotational system, over supports, when different amount of constraint is prescribed.
- If there is a correlation between different load cases. One of the key findings show that the highest load is not necessarily the worst load.

**Further investigations that can be made but require more background**

- The effect of extra stiffness from railings and evaluate if is possible to make the railings more stiff (for example with cross bracing) to get better dynamic performance instead of making the bridge plate thicker?
- It was stated that a "general approach" is hard since there are so many influencing parameters. This indicates that a correlation study between the parameters should be performed.
- Is there a need to test for a load model with crowds if the worst case is for one person running?
- Can a correlation between total mass of the structure and the critical damping be produced in order for this value to be more accurate for every respective case?

# 8

# Conclusion

Although this project is just the starting point in the development of dynamic design in SLT bridges there are a few conclusions that can be made.
The overall purpose was to guide the design process of SLT bridges towards a more sustainable path. This thesis had two main aims to be able to fulfill the purpose. The first one targeted the industry and aimed to create a parametrisized script that can make it easier to design SLT bridges with regards to dynamic response in order to make it more attractive to construct bridges in an environmental friendly material. The other one had a more academic aim and that was to apply this script to be able to study important influencing parameters in the dynamic design process together with how this can be implemented in the design codes. This, to be able to give suggestions for a more sustainable design process.

We have succeeded with creating a functioning Python script that together with Brigade/-PLUS can perform a dynamic analysis for multiple span SLT bridges. This can be used by the industry but also in the further studies on these types of bridge.

The studies include both geometrical variations as well as material property variations. It was difficult to find connections between the parameters and the dynamic response when the material properties were changed. For example, if a property that affect both stiffness and mass was changed, such as making the plate thicker. Therefore, one of the main conclusions from this thesis is that further studies need to be made on the dynamic behaviour as material properties are altered.

The key findings of the study were:

- An increase of 1 percent unit can be made for the critical damping ratio which will make a significant difference in the dynamic design
- A span-to-span ratio of 1 is ultimate for longer bridges with a span-to-depth ratio above 30
- The ratio between spans are of higher importance as the total length of the bridge increases.
- Further studies need to be made primarily on how the material parameters affect dynamics rather than the geometry.

In a society where a sustainable development is so highly prioritized it is curious that so much is still as conservative and unchangable. We propose to start with a more thorough dynamic design process as a demand for light-weight bridges but also to make the design process more case specific. As one can see from our results, there can be a gain in efficiency if the geometry of the bridge is used in an innovative way.

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# A
# Appendix

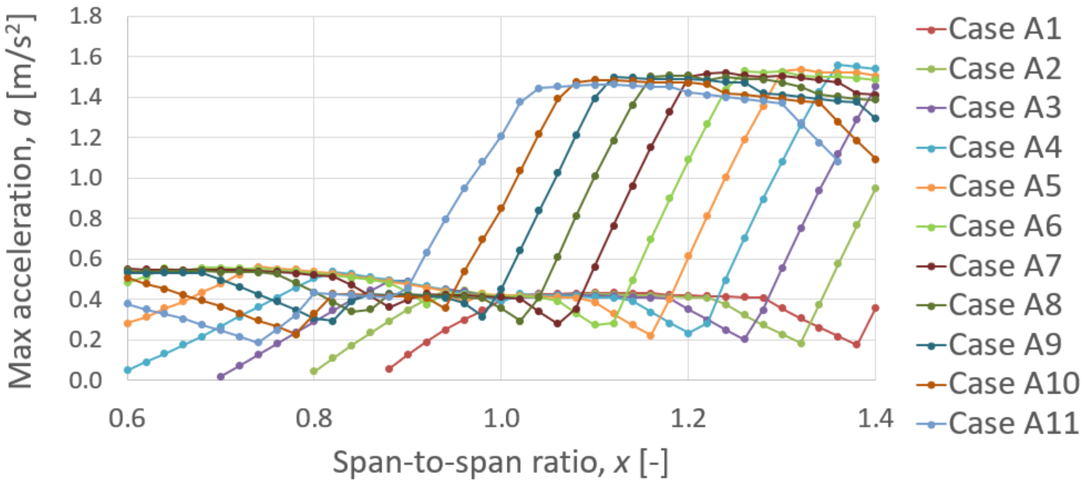


**Figure A.1:** Study A: Span-to-span, compiled results. Explanation of each test can be found in Table 6.2

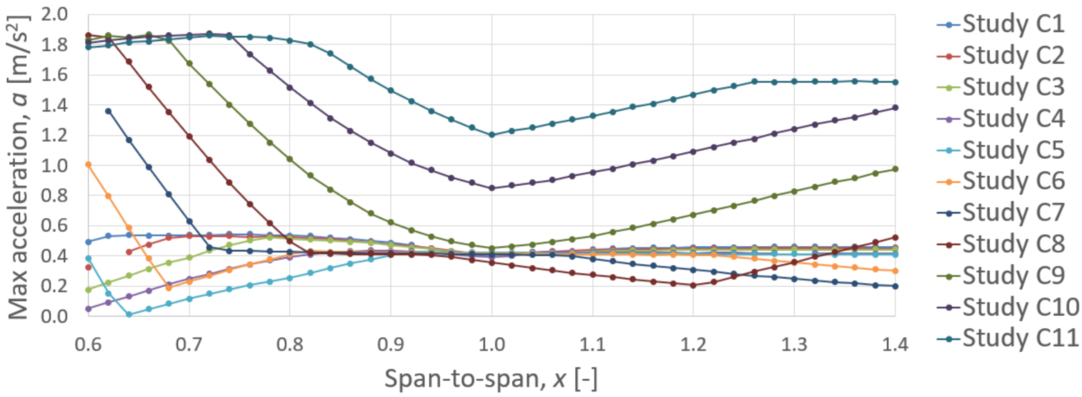


**Figure A.2:** Study C: Support positioning, compiled results. Explanation of each test can be found in Table 6.4

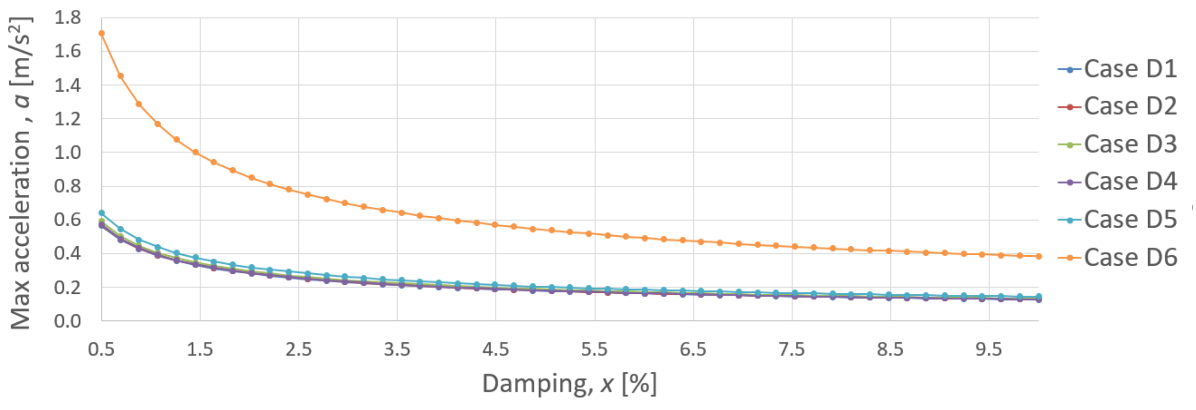


**Figure A.3:** Study D: Damping, compiled results. Explanation of each test can be found in Table 6.5

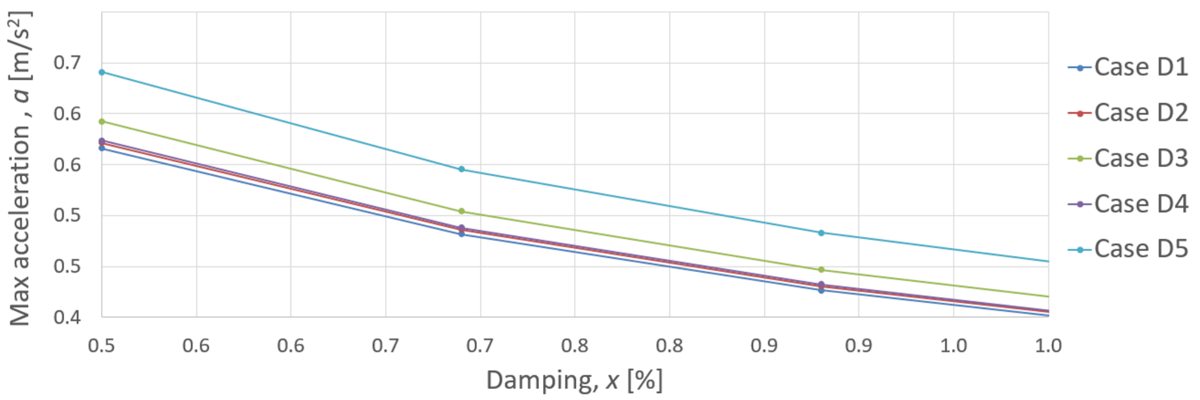


**Figure A.4:** Study D: Damping, case D1 to D5. A piece of the graph i shown to distinguish the different cases. Explanation of each test can be found in Table 6.5

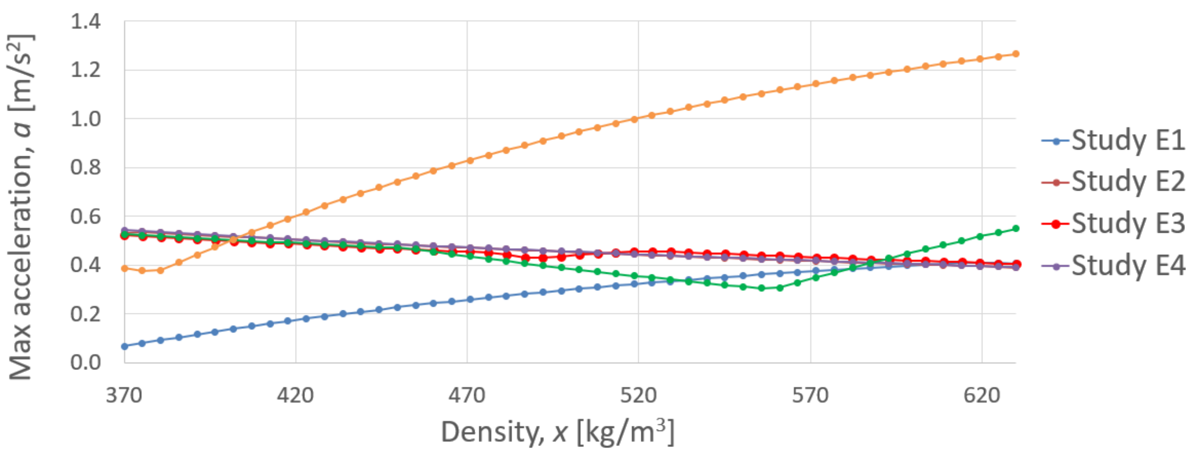


**Figure A.5:** Study E: Density, compiled results. Explanation of each test can be found in Table 6.6