Soil Steel Composite Bridges for High-Speed Railways

2D FEM-ANALYSIS OF THE BJÖRNBO BRIDGE

JOAKIM WOLL
Soil Steel Composite Bridges for High-Speed Railways

2D FEM-analysis of the Björnbo Bridge

JOAKIM WOLL

Master of Science Thesis
Stockholm, Sweden 2014
Abstract

This research aims to analyse the dynamic behaviour of Soil-Steel Composite Bridges when subjected by high-speed trains. The analyses of the dynamic response for these structures are needed since there is little research performed in the present field of knowledge. Since there is also in need to perform separate dynamic analysis for these structures to verify their dynamic response, the dynamic behaviour must be analysed. The research are performed in 2D FE-models in the commercial FE-program Brigade/PLUS since there is of interest to analyse if simplified 2D-models can predict the dynamic behaviour for these structures and verify against design criterions in regulatory documents.

The research is performed by calibrating a reference model against collected field measurements from a constructed Soil-Steel Composite Bridge, SSCB, located in Märsta, Sweden, Märsta Bridge. The calibration process was made to ensure satisfactory results before continuing the research by analysing a future planned SSCB in a case study that is known to in the future be subjected by high-speed trains. The future planned bridge is the Björnbo Bridge located in Skutskär, Sweden. A static structural design is first made with existing methods to verify Björnbo Bridge for static load cases. Attempts is made to verify the Björnbo Bridge against dynamic criterions available in Eurocode documents and Swedish Transport Administration regulatory documents, which includes verifying accelerations limits for 10 different high-speed trains. Smaller analysis of fatigue for the Märsta Bridge and the Björnbo Bridge was also made to verify dynamic stresses from giving fatigue damages.

Since the research is limited for SSCB for dynamic cases, parametric studies are performed for certain parameters identified from an international literature review of earlier studies in both static and dynamic analysis. The studied parameters are: Soil cover depth, Young’s modulus for engineered backfill and different profiles impact. These parametric studies are made to be able to understand influence and sensitivities from the analysed parameters with the long-term goal to develop analysis methods and verifications for SSCB for dynamic load situations.

The calibrated reference model showed that there are difficulties in calibrating acceleration levels that agrees with the field measurements from Märsta Bridge. The expected result from the analysis of Björnbo Bridge was to fulfil static structural design criterions and that the acceleration limits were below serviceability criterions for dynamic analysis according to Eurocode documents. Moreover, that the stresses did not give fatigue damages. From parametric studies, it has shown that the governing parameter is the Young’s modulus for engineered backfill, which affects estimated accelerations in a fashion that not was expected in the beginning. The presumption to perform dynamic analysis with 2D FE-models has shown that all aspects that is needed to verify cannot be performed, such as bending in two directions or twisting mode shapes. Thus, there is in need to find ways to perform dynamic analysis for SSCB with efficient 3D-models.

Keywords: Soil-Steel Composite Bridge, SSCB, Dynamic analysis, Björnbo Bridge, High-Speed Railways, FEM, Soil-Steel culvert, High-Speed Railway Soil-Steel Composite Bridge
Sammanfattning

Denna avhandling syftar till att undersöka det dynamiska beteendet hos rörbroar när dem belastas med höghastighetståg. Analyser av den dynamiska responsen för dessa konstruktioner är behövlig då det finns lite forskning som utförts inom kunskapsområdet. Då man även behöver genomföra separata dynamiska analyser för dessa konstruktioner för att verifiera deras dynamiska beteende, så är det ett behov av att dess dynamiska beteende analyseras. Undersökningen är genomförd med FE-modeller i 2D i det kommersiella FE-programmet Brigade/PLUS då det är av intresse att analysera om förenklade 2D-modeller kan förutse det dynamiska beteendet för dessa konstruktioner och verifiera konstruktionen mot kriterier ställda i styrende dokument.


Då forskningen är begränsad gällande dynamiska studier för rörbroar, så utförs även parametrika studier för parametrar identifierade från en internationell litteraturinventering av tidigare studerade fall för rörbroar gällande både statiska och dynamiska analyser. Dom studerade parametrarna är; Överfyllnadshöjd, Jordmodul för kringfyllning och olika profilers inverkan. Dessa parametrika studier är utförda för att förstå influens och känsligheten i dessa parametrar med det långsiktiga målet att utveckla analysmetoder för att verifiera rörbroar även för dynamisk situationer.


Nyckelord: Rörbro, Dynamisk analys, Björnbo rörbro, Höghastighetståg, FEM, Höghastighetståg rörbro, Dynamisk analys rörbro
Preface

The research that is presented in this master thesis is the final examination from the Master’s program Civil and Architectural Engineering, 120 credits, from the Royal Institute of Technology, KTH. Whereas the research in this master thesis constitutes of 30 credits. The research has been performed at the company Sweco in Stockholm, Sweden, and together with supervision from KTH.

My sincere appreciation goes to my supervisors from KTH: Tech. Dr. Andreas Andersson and Adj. Professor Lars Pettersson for great support during and reviewing of this thesis. Your guidance, enthusiasm to the topic of this thesis and helpful comments has been a great help during this research and helped me finalizing my studies at KTH. In addition, I send my appreciation to Professor Raid Karouni, Head of Division of Structural engineering and Bridges at KTH, for his enthusiasm regarding structural dynamic and helping me choosing my topic for master thesis.

I would like to thank my supervisors at Sweco, Andreas Sjölander and Jacob Hellgren for great comments, discussions regarding results and continuous support and reviewing throughout this thesis. Your help has been invaluable. In addition, I would like to thank Thomas Brutar for allowing me to write my master thesis at Sweco.

I would like to thank Johan Kölfors at Scanscot Technology for the borrowing of licenses to Brigade/PLUS – Without your help; this thesis would not have been possible.

Acknowledgement is given to SIS Förlag for allowing copying of figures from the Eurocode to use in this thesis.

Finally, I wish to give my deepest appreciation to my family, my mother Annika, my father Peter and my brother Markus. Your encouragement through my education has been invaluable. Your support has always given me the strength to keep pushing for my goals.

Stockholm, August 2014

Joakim Woll
Notation

Abbreviations

UIC International Union of Railways
SSCB Soil-Steel Composite Bridge
AASHTO American Association of State Highway and Transportation Officials
DAF Dynamic Amplification Factor
DOF Degree Of Freedom
HSLM High-Speed Load Model
MDS Maximum Design Speed
SLS Serviceability Limit State
ULS Ultimate Limit State
FLS Fatigue Limit State
Mtpa Million Tonnes Per Annum
CHBDC Canadian Highway Bridge Design Code
FEM Finite Element Methods
BEM Boundary Element Methods
RÖK Top height of railway track
LVDT Linear Variable Differential Transform

Latin notation

A Area
D Diameter of the SSCB profile at the spring line
H Distance from the spring line, i.e. widest diameter, to the crown of the SSCB profile
RT Load from a real train
h Height of the cover depth including ballast from the top of the arch to the top surface of the railway sleepers
v The highest allowed speed
p Pressure at soil surface
k Spring stiffness
d Deflection
n0 Fundamental frequency of analysed structure
ni Resonance frequency
vi Resonance speed in
L Determinant length, the length of the influence line
hc Soil cover depth
hc,red Reduced soil cover depth
ydyn Largest dynamic response in terms of deformation
ystat Largest static response in terms of deformation from a real train load or HSLM-A
kyy Interaction factor for flexural buckling about y-axis suggested by the Eurocode
Characteristic yield strength steel

Design yield strength steel

Characteristic ultimate stress per bolt

Characteristic ultimate stress for steel

The amount of cycles with the stress range $\gamma_{ey} \Delta \sigma_i$

Load from HSLM-A

Top radius of SSCB profile

Corner/Side radius of SSCB profile

Bottom radius of SSCB profile

Bending moment, calculated separately for soil- and traffic load, notations used from report 112

Design axial force, notations used in Eurocode

Design bending moment about y-axis, notations used in Eurocode

Largest dynamic response in terms of moments from the finite element solution, should be analysed for both positive and negative moments

Largest static response associated with an arbitrarily point on the analysed structure member from HSLM-A, should be analysed with corresponding positive and negative moment from a static load

Characteristic value for bearing capacity for normal force

Characteristic value for bearing capacity for bending moment about y-axis

Maximum value of the load model that is subjected to bridge

Design shear capacity per bolt

Design bearing capacity per bolt with regard to resistance near a bolthole

Design tension capacity per bolt

Design shear force per bolt in ULS

Design tension force per bolt in ULS

Nominal stress area per bolt

Young's modulus for the steel SSCB-profile

Young's modulus of soil from report 112, method B

Lifetime regarding fatigue (in cycles) for the corrected S-N-curve for the stress range $\gamma_{ey} \Delta \sigma_i$
$D_{d,\text{year}}$  Total cumulative damage for a whole year
$M_{m}^{MN}$  The mass matrix associated with material $m$

**Greek notation**

$\gamma_{M}$  General partial coefficients, suggested by the Eurocode
$\gamma_{Mi}$  Specific partial coefficients, suggested by the Eurocode
$\nu$  Poisson ratio
$\phi$  DAF suggested by Eurocode
$\phi_{\text{red}}$  Reduced DAF suggested by Eurocode
$\phi_{\text{dyn}}$  DAF for general dynamic behaviour
$\phi''$  DAF regarding track imperfection
$\lambda_{f}$  Combined stiffness ratio from report 112
$\chi_{y}$  Flexural buckling about $y$-axis suggested by Eurocode
$\delta_{\text{crown}}$  Estimated crown displacement during backfilling
$\Delta\sigma_{E,2}$  Stress range in pure tension
$\Delta\tau_{E,2}$  Stress range in pure shear
$\Delta\sigma_{c}$  Category number as suggested details specified in Eurocode
$\Delta\sigma_{p}$  Peak to peak stress in Lambda method
$\Delta\sigma_{i}$  Dynamic stress signal in Palmgren-Miner rule
$\alpha_{\text{train}}$  Amplification factor for trainloads suggested by Trafikverket
$\alpha$  Speed dependent factor
$\zeta$  Critical damping as suggested by the Eurocode
$\Delta\zeta$  Additional critical damping as suggested by the Eurocode
$\xi_{\alpha}$  Critical damping fraction used in mode $\alpha$
$\xi_{m}$  Critical damping fraction for material $m$
$\phi_{\alpha}^{M}$  Eigenvector of mode $\alpha$
$m_{\alpha}$  Generalized mass with mode $\alpha$
$\lambda$  Bogie distance from the HSLM-train in analysis
$\omega$  Natural circular frequency of the external load
$\omega_{n}$  Natural circular frequency of the structure
# Contents

Abstract ........................................................................................................................... i

Sammanfattning ........................................................................................................ iii

Preface ......................................................................................................................... v

Notation ....................................................................................................................... vi

1 Introduction ........................................................................................................ 1
   1.1 Background .................................................................................................... 1
   1.2 Aim and scope ............................................................................................ 2
   1.3 Methodology ............................................................................................... 2
   1.4 Limitations .................................................................................................. 4
   1.5 Thesis structure........................................................................................... 5

2 Soil-Steel Composite Bridges for high-speed railways ................................... 7
   2.1 High-speed railways.................................................................................... 7
   2.2 Soil-Steel Composite Bridges - Previous static studies ......................... 9
   2.3 Soil Steel Composite Bridges - Previous dynamic studies.................... 11

3 Design of Soil-Steel Composite Bridges ....................................................... 15
   3.1 Static design principles ............................................................................ 15
   3.2 Dynamic design for high-speed trains ...................................................... 21
      3.2.1 Eurocode ........................................................................................... 21
      3.2.2 Fatigue Limit Analysis ..................................................................... 26
      3.2.3 Swedish design regulations ............................................................. 28
      3.2.4 Finite Element Procedure ................................................................ 29
   3.3 Model calibration of the Märsta Bridge .................................................... 31
   3.4 HSLM-A2 analysis for Märsta Bridge – Dynamic analysis ................... 42

4 Case study of the Björnbo Bridge .................................................................. 51
   4.1 Properties of the bridge ............................................................................ 51
   4.2 Static design ............................................................................................... 55
5 Parametric study on the dynamic performance of SSCB
  5.1 Static design ................................................................. 67
  5.2 Dynamic design ........................................................... 70
  5.3 Parametric study of the dynamic response ....................... 71

6 Discussion and conclusion .......................................................... 77
  6.1 Model insecurities .......................................................... 77
  6.2 Discrepancies: Märsta Bridge and Björnbo Bridge ..................... 77
  6.3 Model calibration .......................................................... 78
  6.4 Dynamic analysis – Björnbo Bridge ...................................... 78
  6.5 Parametric studies on dynamic load cases for SSCB-structures ..... 79
    6.5.1 Influence from parameters in dynamic analysis .................. 79
    6.5.2 Dynamic Amplification Factors ...................................... 80
  6.6 Future research ........................................................... 81

Bibliography ................................................................................. 83

A Static design of Björnbo Bridge ................................................. 87

B Parametric studies – Static load case ........................................ 107
Chapter 1

Introduction

1.1 Background

The name Soil-Steel Composite Bridges is designated to the bridge type that consists of a buried structural system with a helically shaped circular-, elliptical- or vault-shape structure and engineered backfill to give support to the structure. Which make it possible to refer it to a composite structure when the bridge is completed. An example of a typical closed elliptical SSCB is presented in Figure 1.1.

![Figure 1.1: A typical example of SSCB in use for railway-lines. [34]](image)

The definition of a bridge according to Swedish design codes is all spans greater than 2m. Thus, Soil-Steel Composite Bridges, abbreviated SSCB, can be included in this definition if they fulfil the criteria. In Sweden, the usage of the SSCB is mainly for roads and for some cases for railways. The reasons for this is the fast installation time and low production-cost when compared to a portal frame bridge which is another commonly used bridge type for shorter spans. According to the Swedish maintenance system for civil engineering structures, BaTMan [1], there are currently 61\(^1\) SSCB in

\(^1\) A remark should be that this only regards SSCB-profiles manufactured with steel.
Sweden today that is in use as a structural system for railway-lines whilst there are approximately 2900 SSCBs that is in use as a structural system as a passage for roads.

Static design methods for SSCB have been developed at The Royal Institute of Technology, KTH. However, with the increasing demand and development of a high-speed railway network the dynamic behaviour of SSCB must be studied. Since the case is now that such methods does not exist, the dynamic behaviour of SSCB have been analysed by several authors by performing field measurements and finite element analysis.

1.2 Aim and scope

This thesis presents a case study for the future planned SSCB named Björnbo Bridge, that should be designed for high-speed railway traffic. These methods, regarding finite element analysis, are parts of a scientific process with the long-term goal to propose workable methods for the everyday design engineer to evaluate SSCB for design situations where dynamic load cases could be apparent.

There has also been an interest to conduct a parametric study for SSCBs that is subjected by high-speed railway traffic after performing a literature review to see if similar design situations could be found. However, such similar situations have not been found and therefore the thesis that is presented can be regarded as a step in the direction to start developing methods to evaluate this type of bridge for high-speed railway traffic. This research is performed to analyse the influence and sensitivity of a set of parameters.

Moreover, there is an interest to analyse whether or not there is possible to analyse SSCB with 2D FE-models to reduce computational effort from computers.

1.3 Methodology

- International literature review;

A literature review has been performed to see if earlier dynamic cases has been treated for SSCB. The aim has also been to find vital parameters to analyse if they have an effect on the dynamic response. A list of certain phrases/search words, presented below, was determined in advance to use when searching for relevant reviewed articles/reports in scientific journals available on the internet.

- High-speed railway traffic
- Soil-steel culvert
- Soil-steel composite bridge
- Dynamic analysis soil-steel culvert
- Dynamic analysis high-speed railway traffic
- High-speed railway traffic soil-steel culvert
1.3. METHODOLOGY

- **Structural design of Björnbo Bridge:**
  
The static response is simulated with methods developed at The Royal Institute of Technology from the handbook ‘Design of soil steel composite bridges’ 4th edition, in this thesis referred to as report 112, with methods implemented from Eurocode 3 regarding flexural buckling of the crown and bolted joints. The methods are valid up to speeds of 200 km/h for the equivalent trainload involved in the analysis. At hand are preliminary drawings of the Björnbo Bridge, therefore it is needed to perform a structural design calculation to determine the final steel plate thickness for the corrugation.

- **Dynamic analysis – FE-model simulations:**
  
The dynamic response for Björnbo Bridge is analysed with FE-models created in the commercial FE-program Brigade/PLUS. The simulations involve train speeds up to 300 km/h for ULS- and SLS-criterions. A calibration process for Märsta Bridge is performed from earlier field measurements from another thesis project performed by Mellat [3] for the X52-train and HSLM-A2 train. The FE-simulations also involves separate FLS-analysis that is needed to be performed for SSCBs.

- **Parametric studies for SSCB:**
  
  Parametric analysis are performed with methods in report 112 and in FE-models for several configurations which involves the vital parameters identified in the literature review. The parametric analysis is performed to analyse the difference in structural response for SSCBs and influence of the chosen vital parameters may give.

The main topics this thesis investigates can be summarized in the presented list below;

- Vital parameters from an international review
- Parametric analysis with static/dynamic design methods
- Comparison from calculated results from FEM-simulations and field measurements
- Influence of vital parameters on the static response by means from report 112 and on the dynamic response by means of finite element solutions
- Dynamic Amplification Factors for displacements and moments
- Investigate the effect on resonance speed for the different parameters under investigation
- Fatigue limit assessment based on static procedures with the Lambda method and dynamic stress-time history with the use of the Palmgren-Miner rule on cumulative damage for the bolted joints located at the crown of the SSCB.
1.4 Limitations

Since this thesis involves the future planned² Björnbo Bridge in Skutskär, the structural parameters (corrugation etc.) have been chosen by strictly following preliminary drawings of the structure.

Since the name SSCB gathers many different types of soil-steel composite bridges. A decision has been made to only analyse two profile types in different span lengths provided from Viacon’s standard profiles in a parametric study. Only profiles manufactured in steel will be analysed.

The dynamic methods described in Eurocode are derived from simply supported structures, which make that they are not directly applicable for SSCB-structures. For the designers today there is a need of making good engineering judgements of how these methods should be applied in the case of SSCB-structures. In this thesis, some assumptions are made outside the framework suggested by Eurocode which are made in discussion with supervisors and is clearly stated where such assumptions is made.

The case for SSCB structures is that they are both a wave propagation problem and a structural dynamics problem when the analysis is performed with finite element approximations. Wave propagation problems, which is generally created by blast or impact loading and high-frequency content. This involves that more modes needs to be considered when acceleration as function of time is wanted. The other type is called Structural dynamics problems, as an example structures subjected by earthquake loads. The response is dominated by lower modes, thus low-frequency content. The response is analysed for several periods of the lower frequencies. [31]

In this thesis, the SSCB is dealt with as a structural dynamics problem, which leads to that the computational routine used in FE-analysis is limited to modal analysis.

Damping is the term that contributes with energy dissipation, which causes the amplitude of a free vibration to decay with time. The damping will be modelled as described in Eurocode documents in this thesis.

An attempt in this thesis is made to analyse SSCB in 2D-equivalent models with linearly elastic presumptions. Thus, it is important to find proof from other sources to validate the results obtained from the approximated solution method, such as analytical methods with known solutions or field measurements. This is not an easy task since there are few studied cases for SSCB linked together with high-speed railway traffic and thus there are few analytical examples and field measurements is very expensive and is not a suitable method for building new civil engineering structures.

² Parallel with this thesis a real live design project is conducted for the Björnbo Bridge. However, the real live design project is not connected with this thesis project by any means.
1.5 Thesis structure

The references used in this literature are marked with brackets and is placed adjacent to the literature that is referred. If a reference regards a whole paragraph, the bracket is placed at the end of the paragraph. In advance of the thesis, a list of used notations used in equations is found, this is to contribute a pleasant reading and not disrupt the flow of the text in the thesis. For some cases where the notation used is of high importance, it has been placed directly after where it is used with a description of the notation. Reference given to Eurocode documents in the body of this thesis is given to the full document name and the chapter of which information is taken. For some cases it has been more convenient to refer to tables or figures presented in Eurocode, but should not be confused with the tables and figures given in this thesis. The standard principal in the body is following the example given below.


Chapter 2 presents a international literature review describing the field that collects the current high-speed railway network in the European region. Some previous research of both static and dynamic analysis is also presented with a short description of each research and the conclusions from each research article.

Chapter 3 present the current available static structural design method that are applicable for SSCB, i.e. report 112, and dynamic design methods taken from the Eurocode documents that are analysed in FE-models. Moreover, a thorough description of how the discretized 2D-model is created in Brigade/PLUS for a reference model that is later used to simulate the behaviour of Björnbo Bridge.

Chapter 4 presents the results from the case study of Björnbo Bridge with description of the structure and the result from the static design methods and results from the dynamic FE-simulations with comments.

Chapter 5 presents the parametric studies that are performed for several parameters that have been identified from the literature review. The results from the parametric studies is also presented with comments.

Chapter 6 presents a discussion of the validity of the results and conclusions drawn from the results, which is confined to the dynamic analysis. Some suggestions for further research are presented.

Appendix A presents a limited version of the static structural calculation of Björnbo Bridge.

Appendix B presents the parametric analysis performed with methods from report 112.
Chapter 2

Soil-Steel Composite Bridges for high-speed railways

In the forthcoming chapter, a presentation of the increasing high-speed railway traffic is presented, chapter 2.1. Some previous research are presented for field measurements and FE-analysis in 2D/3D for static load cases which may contribute to further investigations of modelling discretized properties correctly, chapter 2.2. Some literatures that also treat dynamic behaviour of SSCB is presented. These studies are important to fulfil some of the knowledge gap of dynamic behaviour for SSCBs and resembles the main topic of the research presented in this thesis, chapter 2.3.

2.1 High-speed railways

The definition of high-speed railway traffic is a railway line where the train travels with a speed at least of 250 km/h according to UIC [4]. In 1964, the first high-speed rail transportation could be found between Tokyo and Osaka; it had a functional speed of 210 km/h and was later upgraded to 270 km/h [4].

Since then, more high-speed railways have been developed both in the existing and in the future planned railway network. In general, the tendency is so that the high-speed railway traffic is increasing in the existing railway network and the future planning. This is due to a demand to increase transport efficiency and provide faster transportation for customers choosing railway traffic.

In the European region [4], the high-speed railway traffic has been active for about 40 years. The first high-speed railway was built in Italy in 1970 with a speed of 200-250 km/h. In France, the first high-speed railway was built 1981 with a speed of 260 km/h. The high-speed railway traffic is widely spread in Europe today as can be seen in Figure 2.1.
Figure 2.1: High-Speed Railway traffic in Europe 2013. A red line indicates a railway line with high-speed in operation. A red dotted line implies that high-speed is in development. Green lines indicate that upgrading to high-speed railway of the railway section is in operation.

UIC [9] has gathered information about the current high-speed railway network around the world. Table 2.1 shows a presentation of each continent's current high-speed railway network that either is in operation, under construction or planned.

Table 2.1: High-speed railway infrastructure in the world.

<table>
<thead>
<tr>
<th>Continent</th>
<th>In operation (2013) [km]</th>
<th>Under construction [km]</th>
<th>Planned [km]</th>
<th>Total (2025) [km]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Europe</td>
<td>7 378</td>
<td>2 565</td>
<td>8 321</td>
<td>18 264</td>
</tr>
<tr>
<td>Asia</td>
<td>13 732</td>
<td>11 199</td>
<td>6 258</td>
<td>31 190</td>
</tr>
<tr>
<td>Africa</td>
<td>-</td>
<td>200</td>
<td>480</td>
<td>680</td>
</tr>
<tr>
<td>South-America</td>
<td>-</td>
<td>-</td>
<td>511</td>
<td>511</td>
</tr>
<tr>
<td>North-America</td>
<td>362</td>
<td>-</td>
<td>777</td>
<td>1 139</td>
</tr>
<tr>
<td>World</td>
<td>21 472</td>
<td>13 964</td>
<td>16 374</td>
<td>51 784</td>
</tr>
</tbody>
</table>

In Sweden today, there are some high-speed lines in operation. Botniabanan between Umeå-Nyland [20] is designed for speed up to 250 km/h; the current traffic consists of both passenger and freight trains. The west coast-railway between Gothenburg-Malmö [21] was recently upgraded to speeds up to 250 km/h. The traffic consists of both passenger and freight trains and fulfills an important link for long-distance traffic.

---

A remark should be that the speed in each country varies in-between 200-360 km/h in Table 2.1.
between Copenhagen-Malmö-Gothenburg-Oslo. The eastern-link [22] between Stockholm-Linköping is one of the on-going high-speed projects in Sweden that is going to be subjected by high-speed railway traffic, speeds up to 320 km/h. The increased speeds along the railway network leads to the conclusion that the civil engineering structures designed today might be exposed to higher speeds in the future. This also leads to the conclusion that the civil engineering projects conducted today might need to consider this when designing civil engineering structures.

In Table 2.2, the current high-speed network in Sweden is presented with complementary information to UIC [9] from Trafikverket with railway information for section between Stockholm-Linköping and Umeå-Nyland. A star (*) indicates that the railway line still is in the planning stage of design.

<table>
<thead>
<tr>
<th>Section</th>
<th>Maximum speed [km/h]</th>
<th>Distance [km]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Malmö-Gothenburg</td>
<td>250</td>
<td>750</td>
</tr>
<tr>
<td>Stockholm-Linköping*</td>
<td>320</td>
<td>150</td>
</tr>
<tr>
<td>Umeå-Nyland</td>
<td>250</td>
<td>190</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>1090</td>
</tr>
</tbody>
</table>

Since high-speed railway traffic projects is developing in Sweden, there is also a need of building suitable civil engineering structures that can sustain the loads that it will be exposed to from high-speed traffic. Since there are many bridge types that can fulfil this purpose, it may seem that there is no problem to find a suitable bridge type. However, a bridge type that may be suitable for a static load case may be un-suitable for a dynamic load case or vice versa. Therefore, it is important to have design methods to analyse both static and dynamic load cases.

### 2.2 Soil-Steel Composite Bridges - Previous static studies

The main usage of SSCB is for road and railway bridges in Sweden. The usage of SSCB internationally has been found in countries such as Canada, Poland, America and Turkey. Where some research articles have been found that treats the subject of SSCB. Some information in design codes such as CHBDC [11] has also been found, but is not presented in this thesis. The topics from the research articles mainly treat field measurements, FE-modelling in 2D/3D for static load cases for SSCB.

El-Sawy [15] addresses the issue of the drawback of analysing SSCB with 2D FE-models and investigates 3D FE-modelling of the Deux Rivieres circular SSCB and the Adelaide elliptic SSCB with ANSYS software. The subjected loads to the SSCB are truckloads, modelled as vertical concentrated point forces. The soil material is assumed to behave linearly elastic and the analysis only considers the external load of the truck, i.e. no additional load from soil pressure. The soil modulus is assumed to vary and thus two values are analysed, 30 MPa and 80 MPa. The steel plate of the SSCB is analysed for two cases for comparison purposes, namely isotropic and orthotropic behaviour.
The analysis performed by El-Sawy is connected to field measurements to compare the FE-results with.

The results presented by El-Sawy showed that the model with the isotropic steel plate gave smaller displacements than the orthotropic. The axial forces show overestimations for the case of using an orthotropic steel plate when analysing Deux Rivieres SSCB and underestimations while analysing Adelaide SSCB relative to the field measurements. In the longitudinal direction of the pipe, the orthotropic steel plate show negligible axial forces while the case of the isotropic steel plate showed unrealistic high values.

Flener [41] performed research with field instrumentations for the long-span steel arch railway bridge constructed in Skivarpsån, Sweden, which resulted in three reports. Of which the last and final report, Part III, is referred as Flener [41] in this thesis.

In Flener [41], it is describe how Skivarpsån was instrumented with strain gauges, in both valley and crest of the corrugation, which measure strains in the longitudinal direction of the bridge. Furthermore, displacements were measured at the crown of the bridge with LVDT equipment. Skivarpsån is a single radius arch shape with a span of 11.2m with a SuperCor S37 corrugation. The tests were performed with the RC4 locomotive with four axles and a total service weight of 78 tonnes. Static, dynamic and braking tests were performed for Skivarpsån. The static tests were conducted for 10 different loading positions of the locomotive.

The differences in static and dynamic displacements that were observed during the field measurements were small. Thus, it was concluded that displacements would not be an issue for the Skivarpsån Bridge. The dynamic moments were higher than the corresponding static moments from the loading positions. The axial forces for the dynamic tests also present higher values than the corresponding static tests. However, there was not possible to state a clear conclusion for axial forces. Since the main difference only was visible when the first axle was above the crown and the speed were increased for the dynamic tests.

Abdel-Sayed and Salib [12] analysed the minimum soil cover depth for SSCB when the spans is increasing above 7.6m according to CHBDC methods [11]. The analysis was performed with plain-strain FE-analysis with the FE-program ABAQUS. The applied load was truckloads from AASHTO documents. The study involved SSCB with spans up to 15.2m for circular SSCBs and 21.3m for arches with deep corrugations.

Conclusions from Abdel-Sayed and Salib are that the corrugation depth has a considerable stiffening effect when analysing minimum soil cover depth that is needed for larger spans. Mainly, the effect is visible for circular SSCBs below spans of 10.7m. When the circular SSCB span is increased above 13.7m and the arch is increased above 19.8m, the stiffening effect of the corrugation is not visible.

Sutubadi and Khatibi [41] performed FE-analysis in the commercial FE-program PLAXIS and studied the variation of soil properties in parametric studies with 2D FE-models according to Mohr-Coulomb theory. The investigated soil parameters was;

---

4 Presented in chapter 2.3.
cohesion, Young’s modulus (30-90 MPa), the internal friction angle, Poisson ratio for soil, and dilatation angle. The analysed SSCB was the Deux Rivieres circular SSCB subjected by truckloads.

The conclusions from the research performed by Sutubadi and Khatibi are that by increasing the cohesion of the soil leads to an increase in the SSCB strength. By increasing the Young’s modulus of the soil, it increases the amount of loading the structure can withstand before failure occurs. The changes of the friction angle have minor changes on the total stability and stiffness of the steel plate. Increasing of the Poisson ratio give that the plastic index of the soil increases and thus the stability provided by the soil slightly decreases. The changes of the dilatation angle have negligible effect on the stability of the SSCB.

Yeau et al. [14] conducted parametric studies for different soil cover depths. Their conclusion was that the normal forces and deflections decreased with an increased soil cover depth. It was also presented that crown deflections depends more on SSCB shape than the size of the SSCB, i.e. span length.

2.3 Soil Steel Composite Bridges - Previous dynamic studies

Dynamic tests of SSCB
Yeau et al. [13] evaluated dynamic loading from a passing truck at varying speed (8-64 km/h) and studied the effect of various parameters. It was observed that the maximum measured deflections in the field observation were lower than the corresponding static loading from the truck. It was concluded that the SSCB deflection decreased nonlinearly with increasing soil cover depth. Under dynamic loading, deflections and strains increased significantly when soil cover depth was less than 0.9m.

Beben [16] analysed DAFs from the passing of a truck in varying speeds (10-70 km/h) over a closed pipe-arch SSCB. Conclusions from the analysis were that both displacements and strains were higher during dynamic loading than static loading. The conclusion for the factor that gave most influence for the DAF was the SSCB span. When the span increases, the DAF increases.

Beben [17] performed an experimental study of the dynamic impact of service trainloads on two SSCB-structures near each other. The reported conclusions were that the dominant frequencies in the SSCB were below 6 Hz. In this case, the study contained trainloads, but the speeds of the trains were lower (70-120 km/h) than for the high-speed railways.

Research in Sweden on dynamic response for SSCB
Flener [18] performed research on integral bridges and culverts. The research treats the determination of Young’s modulus and compares the methods derived by Pettersson/Sundquist with those derived by Duncan from empirical investigations in 1978.
Flener [18] states that the soil cover depth is directly proportional with the bending moments due to soil load; it was observed that the trend of changing soil load on the SSCB was not consistent with the change in bending moments. Flener stated that the live load, i.e. dynamic loading, is dependent on how the soil-structure interaction is modelled. Models such as Winkler soil model in Eq. (2.1) with elastic springs with constant stiffness is one of the easiest ways of modelling how large the pressure is at the soil surface from deflections and is presented by Flener. Usually a non-linear soil model is assumed more realistic; the behaviour can be estimated from linear elastic parameters with an incremental manner to capture if the behaviour follows a non-linear trend.

\[ p = k \cdot d \]  

(2.1)

Flener [41] performed dynamic tests on the long-span arch steel culvert railway bridge over Skivarpsån, Sweden. The test vehicle was an RC4 locomotive with four axles and a service weight of 78 tonnes when the locomotive was moving in different speeds from 10-125 km/h. Field measurements was gathered and filtered with a 60 Hz low pass filter. The field measurements consisted of measuring displacements with LVDT equipment, strains, axial forces, axial stresses and bending moments. The field measurements was compared to theoretical calculations and it was concluded that the measured moments were about 3 times smaller than the calculated value. However, it is mentioned that the used methods for theoretical calculations applies a safety factor of two to the estimated result.

Emanuelsson and Roland [19] evaluated the determination of the elastic soil modulus for the backfill around the SSCB structure. Their main objective with the research was to come up with new methods to evaluate Young’s modulus. The ambition was to find relationships between the grain size distribution, level of compaction and the mechanical properties of the soil by empirical investigations. Laboratory tests were performed to evaluate Standard Proctor and Modified Proctor.

Field tests were performed to measure elastic modulus of soil with ramming weights, i.e. dynamic methods, at two working sites of an SSCB. The values obtained from the measurements varied between 20-425 MPa. The variation was concluded to depend on the choice of ramming weight-equipment used in the field tests. However, the analyses performed by Emanuelsson and Roland showed that there can be a large variation in the Young’s modulus of soil within small areas adjacent to the pipe.

Comparisons of the measurements were performed against several methods. Emanuelsson and Roland’s analysis was built on earlier empirical investigations. Therefore, their conclusion was that it is hard to determine the relationship between earlier empirical investigations and the development of new methods for the evaluation of elastic modulus for soils.

It can also be noted that Emanuelsson and Roland presented higher measurement values in their thesis of the soil modulus than what could be found in the majority of the other methods used to compare with.
Mellat [3] performed a case study for a SSCB built in Märsta and evaluated the dynamic behaviour with 2D and a 3D finite element model and applied X52-trainloads and HSLM-trainloads.

Mellat also evaluated different parameters influence on the dynamic behaviour. The elastic soil modulus was evaluated as a dynamic modulus that considers short-term effects, and thus can be chosen much higher than an ordinary elastic soil modulus that assumes long-term effects. The response of different soil modulus varied in the investigations. The effect of soil density was observed on the acceleration of the ballast and it was concluded that the effect in the ballast layer and the crown did not vary.

Mellat evaluated the differences between modal analysis and direct integration with implicit methods. It was observed that the modal analysis generated the highest accelerations. The direct integration used with implicit methods, generated result within the acceleration limits for both the crown and in the level of the ballast, the result needed filtering after the result was obtained. A recommendation from Mellat was to confine the analyse methods to direct integration methods since the results in terms of accelerations obtained from modal analysis were not reliable, modal analysis could still be useful when predicting resonance behaviour.

In the 2D-analysis that was performed, natural frequencies for four vertical bending modes, with the fundamental frequency equal to 11.9 Hz, were obtained. For the 3D-analysis, a fundamental frequency of 12.4 Hz was obtained. Thus, some discrepancies in predicted frequencies were observed between 2D- and 3D-models.

Evaluation of DAFs from Mellat by following the Eurocode for carefully maintained tracks were calculated to 1.39 and DAF for carefully maintained tracks for fatigue damages to 1.56. Mellat finally concluded that the Märsta Bridge would be able to sustain high-speed railway traffic if such were the case in the future.

Conclusively, it has been evident during the literature review that there are few cases where research articles on dynamic behaviour for SSCB is analysed, such as the research performed by Mellat.
Chapter 3

Design of Soil-Steel Composite Bridges

In the following chapter the static design principles from Report 112 is presented in a brief summary, chapter 3.1. Eurocode and Swedish design regulation documents that describe dynamic design checks for civil engineering structures are presented in chapter 3.2. The calibration process with the X52-train for the reference FE-model for SSCB is presented in chapter 3.3. A dynamic check analysis with HSLM-A2 train of Märsta Bridge is presented thoroughly in chapter 3.4. The reference model will later be used in chapter 4 to fulfil the purpose of the case study of Björnbo Bridge and in chapter 5 fulfilling the purpose of the parametric analysis of SSCB.

3.1 Static design principles

The theories for the calculation methods used in report 112 is based on two theories developed in 1970s and are only mentioned by their names in this thesis.

- The SCI-method (Soil Culvert Interaction) presented by Duncan (1978) and Duncan (1979), and
- Klöppel & Glock (1970)

The method assumes that the SSCB has a uniform section over a long distance in the longitudinal direction of the pipe and it is assumed that it is possible to consider that the pipe is subjected by a strip with a length of one meter of loading perpendicular to the axis of the pipe. When the SSCB is subjected by traffic load, the top of the arch is regarded as founded on elastic supports equivalent to the total of the lateral support provided from the surrounding soil adjacent to the pipe. The top arch is assumed to be continuously elastically supported with the aid of the supporting mass above. The top of the arch is generally denoted in-between the quarter points of the pipe and is the area generally assumed affected by traffic loads.

The design theories is then adjusted to fulfil the at the time regulatory Swedish national design documents BSK07 and Bro 2004. However, the method is applicable to use in the current design documents, i.e. Eurocode, by applying the partial coefficients suggested from them. The method is created so that it is possible to analyse several
types of soils, profiles, loads etc., which makes it flexible to be used in different design situations for SSCB. The profiles in the method need to fulfil certain criterions coupled together with the shape of the profile. The criterions for the profiles in this thesis, i.e. VT-, VE-profiles, can be found in chapter 1.2.3 in report 112 [29]. The two profile types are presented in Figure 3.1.

\[ \text{Figure 3.1: Viacon profiles under study.} \]

**Tangent modulus**

For the case of determining soil-stiffness, report 112 contains method A and method B. Method A constitutes of a tangent modulus and is regarded as a direct approach without knowing the geotechnical conditions. Method B constitutes of a tangent modulus and is regarded as a method that more correctly describes the geotechnical conditions since it is based on input given from a geotechnical investigation. Both method A and method B assumes long-term effect. In this thesis, method B is the chosen method since geotechnical investigations is available. Thus, method B is the only presented method.

To be able to use this method, the designer needs to have information about:

- Particle size distribution \((d_{10}, d_{50}, d_{60})\)
- Degree of compaction (dry density and maximum dry density)
- Stress level in the surrounding fill calculated using the passive earth pressure at a depth equal to the cover depth plus \(H/2\).

The final equation to determine the design Young’s modulus \((E_{sd})\) of the engineered backfill is presented in Eq. (3.1), the Young’s modulus is calculated at the level of the quarter points. For more details in calculation of characteristic friction angle \((\varphi_c)\), stress levels in the surrounding fill \((\sigma_i \text{ and } \sigma_j)\), modulus ratio \((m)\) and stress exponent \((\beta)\), reference is given to Appendix A.
Report 112 states specific requirements for the soil material requirements adjacent to the pipe, also called engineered backfill. For more information regarding the specific requirements on the engineered backfill, reference is given to chapter 1.2.4 in report 112 [29].

**Combined stiffness of pipe and soil**

The soil and steel pipe is together working in combination when the structure is in use, hence, the name soil-steel composite bridge is denoted for these structures. When determining the stiffness in report 112, it is by considering both of the materials with a combined stiffness parameter according to Eq. (3.2).

\[
\lambda_f = E_{sd} \frac{D^3}{(EI)_s}
\]  

(3.2)

This stiffness parameter affects the design moment from the surrounding soil and the traffic load and is thus important to determine correctly. The stiffness parameter has limits resulting in limitations in the design method presented. The limits of the stiffness parameter are in the range \(100 \leq \lambda_f \leq 50\,000\).

**Soil cover depth**

The recommendation from report 112 regarding soil cover depth is that the height \(h_c\) should always be at least 0.5m for the design method to be valid. For the cases when the SSCB is constructed to fulfil a function for a road or railway bridge, the recommended height\(^5\) \(h_c\) is increased to 1.0m, which should consist of at least 0.5m ballast material. This requirement mainly regards that a required cover depth is maintained during maintenance of the track.

A reduction of the soil cover depth is generally made since it is shown that during the backfilling process the crown of the pipe has a tendency to rise when the soil pressure increases along the sides. The reduction is made according to Eq. (3.3) and the rise of the crown is estimated with Eq. (3.4).

\(^5\) Note that TRVR Bro contains a clause that over-rules this statement, presented in chapter 3.2.3.
\[ h_{c,\text{red}} = h_c - \delta_{\text{crown}} \]  
(3.3)

\[ \delta_{\text{crown}} = 0.013 \frac{\rho D}{E_f} \left( \frac{H}{D} \right)^{2.8} \cdot \lambda_f \cdot \left[ 0.56 - 0.2 \ln \left( \frac{H}{D} \right) \right] \]  
(3.4)

**Dynamic amplification factor suggested in report 112**

The suggested DAF in report 112 is based on that by increasing the soil cover depth, the effect from the live load can be reduced by a factor \( r_d \). The factor is determined with the expression in Eq. (3.5). Note that the factor is determined from the reduced soil cover depth.

\[
\begin{align*}
    h_{c,\text{red}} < 2m \Rightarrow r_d &= 1.0 \\
    2m < h_{c,\text{red}} < 6m \Rightarrow r_d &= 1.10 - 0.05 h_{c,\text{red}} \\
    6m < h_{c,\text{red}} \Rightarrow r_d &= 0.8
\end{align*}
\]  
(3.5)

Report 112 estimate that with increased soil mass, i.e. increased soil cover depth, the dynamic effects decreases due to frictional losses and load spreading in the soil. It can be noted that the method described in report 112 does not include resonance effects from live loads.

**Load model**

The design load for railway bridges according to Eurocode documents is given in Eq. (3.6), SS-EN 1991-2:6.4.6.5(3). Three different load models are available. However, the load that gives largest response should be chosen. The traffic load in report 112 constitutes of the LM 71-train model as presented in Figure 3.2 and is expressed as an equivalent line load, the line load consider if there are a single railway track or a double railway track located on the railway embankment. The equivalent line load is based on Boussinesq and accounts for 3D-effects. The equivalent line load that is used in report 112 is presented in Figure 3.3.

\[ \phi \times F_{\text{design}} \]

\[ F_{\text{design}} = \max(LM71, \ SW/0, \ SW/2) \]  
(3.6)

\[ ^{6} \text{Note that it represents the characteristic load of LM71-train as a function of increased reduced soil cover depth when the soil cover depth is reduced according to Eq. (3.3).} \]
3.1. Static design principles

Load model: LM 71

Load model: SW/0 & SW/2

Figure 3.2: Static load models. SS-EN 1991-2: Figure 6.1 & Figure 6.2 [25]

Figure 3.3: Equivalent line load LM71-train. [29]

Note that the load for single track and double track are the same up to about a reduced soil cover depth of 2m, after 2m of reduced soil cover depth the functions deviates from each other. A limitation in this thesis has been to analyse soil cover depth in the ranges 1.1-3.0m. This shows in Figure 3.3 that the characteristic trainload is underestimated in some portion of the range when performing the parametric studies for a single railway track.

Verifications for SLS

The verifications implemented in report 112 are both from Eurocode 3 and BSK07. The verifications made in SLS regard safety against yielding in the wall of the pipe and settlements in the entire volume of soil that is surrounding the culvert. In this case, control is only performed for yielding in the wall of the pipe and the settlements occurring is assumed negligible and hence not checked by any means. For more information that is detailed regarding SLS verifications, reference is given to chapter 5.2 in report 112 [29].

19
CHAPTER 3. DESIGN OF SOIL-STEEL COMPOSITE BRIDGES

Verifications for ULS

The design normal forces and design moments in ULS for the trainload LM71 are verified with suggested methods provided in Eurocode 3 and BSK07 as is implemented in report 112. The controls is performed for two cases, one with only normal force acting and bending moment equal to zero, the other case is by controlling interaction between normal force and bending moment. The verification made according to BSK07 is made as a comparison value against Eurocode. The controls are made to verify the upper part of the pipe for flexural buckling.

Bolted joints in the SSCB structure are verified in ULS by checking three cases; shear force, tension and interaction for shearing and tension. The verifications are performed for the number of bolts \( n \) per metre width of the culvert in each joint. For more information that is detailed regarding ULS verifications, reference is given to chapter 5.2 in report 112.

Verifications for FLS - Lambda method

The lambda method that is implemented in report 112 in general means that the design stress range is decided from the LM71-train model and then several factors, \( \lambda_1 - \lambda_4 \), is multiplied with the design stress range which amplifies the stress range. The lambda factors considers the following actions and is determined from functions as presented in Figure 3.4.²

![Figure 3.4: λ-factors for railway bridges. SS-EN 1993-2: Table 9.(4,5,6) [36](https://example.com)](https://example.com)

The factor \( \lambda_4 = 1.0 \) as a recommendation from Swedish design regulation documents provided from Trafikverket, TRVFS Ch.19 §17 [37], the factor takes into consideration

² In the graph to the left, there is a difference regarding if the \( \lambda_1 \)-factor regards a mixed traffic or HSLM-trains.
that several tracks are located on the railway embankment. The final lambda factor that is used to amplify the stress range, \( \lambda_{\text{max}} \), is presented in Eq. (3.7), SS-EN 1993-2:9.5.3.

In addition, the stress range amplitude from the applied load, estimated as the simplified peak to peak stress \( (\Delta\sigma_p) \), should be multiplied with the dynamic factor as determined in Eq. (3.9) and a suitable partial safety factor decided by the Eurocode.

The final stress range that is determined as presented in Eq. (3.8) according to SS-EN 1993-2 (2006): 9.4.1 [36].

\[
\lambda_{\text{max}} = \lambda_1 \cdot \lambda_2 \cdot \lambda_3 \cdot \lambda_4 \leq 1.7 \quad (3.7)
\]

\[
\Delta\sigma_E = \lambda_{\text{max}} \phi \Delta\sigma_p \\
\Delta\sigma_p = |\sigma_{p,\text{max}} - \sigma_{p,\text{min}}| \quad (3.8)
\]

In Table 3.1, the verifications is presented with its correlated category number and which detail that is chosen from Eurocode and TRVK Bro to analyse the SSCB for FLS. For more information that is detailed regarding FLS verifications, reference is given to chapter 5.2 in report 112 [29].

<table>
<thead>
<tr>
<th>Stress location</th>
<th>Steel plate at the crown (tension)</th>
<th>Steel plate near a bolted joint (tension)</th>
<th>Bolted joint (Shear)</th>
<th>Bolted joint (tension)</th>
<th>Bolted joint (interaction between shear and tension)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Detail taken from design document</td>
<td>Table 8.1, type (5)</td>
<td>TRVK Bro, J.3.2.2</td>
<td>Table 8.1, type (15)</td>
<td>Table 8.1, type (14)</td>
<td>Table 8.1, type (14) and type (15)</td>
</tr>
<tr>
<td>Category number</td>
<td>( \Delta\sigma_c = 125 )</td>
<td>( \Delta\sigma_c = 71 )</td>
<td>( \Delta\tau_c = 100 )</td>
<td>( \Delta\sigma_c = 50 )</td>
<td>( \Delta\sigma_c = 50, \Delta\tau_c = 100 )</td>
</tr>
<tr>
<td>Unit</td>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[MPa]</td>
</tr>
</tbody>
</table>

### 3.2 Dynamic design for high-speed trains

#### 3.2.1 Eurocode

SS-EN 1991-2:6.4 give suggestion of dynamic effects that should be considered by the designer in a dynamic analysis. In SS-EN 1991-2:6.4.4 [25], a flowchart is at hand to determine between a static or dynamic analysis. The flowchart helps to determine if a full dynamic analysis, with determining bending and twisting modes, is in need. Otherwise, it is sufficient to only perform a static analysis and apply DAFs to increase the static loading. By applying DAFs, all the dynamic effects of the structure are included.
An upper and a lower bound criterion for the frequencies are available to be determined with methods provided in the Eurocode. The criterions are determined from simply supported bridges with longitudinal beam effects or simply tensed slabs with negligible impact on resilient supports. However, since SSCB is denoted as complex structures the methods provided in Eurocode is not applicable. Instead, it is suggested to perform a full dynamic evaluation that includes determining bending and twisting mode shapes and DAFs.

A DAF $\phi$ is decided according to Eq. (3.9) for carefully maintained tracks by following SS-EN 1991-2:6.4.5.2. The length $L_\phi$ is the determining length for the influence line, for SSCB there are no applicable suggestions in Eurocode, SS-EN 1991-2:6.4.5.3. The designer can estimate the determining length by analysing the length of the influence line for deflection for the structural member that is to be designed. For this case, a finite element model is used to determine the length $L_\phi$ and it will be assumed that the track is carefully maintained. Eurocode provides the ability for the designer to reduce the dynamic factor that is calculated in Eq. (3.9) for vault-bridges when the cover depth exceeds 1.0m. The reduction of the dynamic factor is calculated as Eq. (3.10), SS-EN 1991-2:6.4.5.4. However, the reduction is not allowed to implement in the design if more than one track is located at the railway embankment.

$$\phi = \frac{1.44}{\sqrt{L_\phi - 0.2}} + 0.82 \quad (1.00 \leq \phi \leq 1.67) \quad (3.9)$$

$$\phi_{rel} = \phi - \frac{h - 1.00}{1.67} \geq 1.0 \quad (3.10)$$

**Dynamic analysis – Serviceability limit state**

In SS-EN 1990:A2.4.4.2.1 [24], the limits for accelerations in the bridge superstructure are:

- $\gamma_{bt} = 3.5 \text{ m/s}^2$, for ballasted tracks.
- $\gamma_{df} = 5.0 \text{ m/s}^2$, for un-ballasted tracks.

The limits are set to prevent the instability in the ballast and lift of the bearing for the ballasted tracks and for the un-ballasted tracks, respectively. The limits should be fulfilled within a frequency range to the highest of $[30, 1.5f_1, f_3]$ Hz, where $f_1$ and $f_3$ is the frequencies of the first and third bending mode. Since there are no recommendations for where the acceleration of a SSCB should be analysed, the acceleration limits will solely be analysed with the crown of the SSCB-structures and the ballast-level. This assumption is taken with respect to analysing ballast instability and control the acceleration limits occurring at the crown of the SSCB, which can be denoted the superstructure of SSCB.

According to SS-EN 1991-2:6.4.6.3.2(2)P, two ultimate cases should be analysed when estimating the mass involved when performing a dynamic analysis. The differentiation
will be regarding the ballast and is presented in Table 5.5. Only maximum and minimum values are analysed.

Table 3.2: Differentiation of ballast density.

<table>
<thead>
<tr>
<th></th>
<th>High-density ballast</th>
<th>Low-density ballast</th>
</tr>
</thead>
<tbody>
<tr>
<td>(N/m$^3$)</td>
<td>(N/m$^3$)</td>
<td></td>
</tr>
<tr>
<td>2100</td>
<td>1700</td>
<td></td>
</tr>
</tbody>
</table>

The stiffness, Young’s modulus, of the SSCB and the soil affects the resonance speed. If the stiffness is over-estimated, it will also lead to an over-estimation of the resonance speed. Eurocode does not provide any methods for SSCB structures on how to estimate the Young’s modulus. Since the SSCB is a composite structure with a combined stiffness of the steel pipe and the soil surrounding, both will provide stiffness to the structure.

Maximum dynamic response in structures is highly dependent on the level of damping that the designer can account for in the design. SS-EN 1991-2:6.4.6.3.1(3) gives some suggestions on how to estimate damping for casted beams, reinforced concrete, prestressed concrete, steel and composite structures. The damping suggested from Eurocode assumes linear damping as can be seen in Figure 3.5.

![Figure 3.5: Estimated damping in structures. SS-EN 1991-2: Table 6.6 [25]](image)

If the case is so that a span is shorter than 30 m, Eurocode allows the designer to increase the damping according to Eq. (3.11) or perform a dynamic analysis of the interaction between vehicle and bridge. This is due to that the interactive mass effects between vehicle and bridge can reduce the maximum response when resonance occurs.

\[
\Delta \zeta = \frac{0.0187L - 0.00064L^2}{1 - 0.0441L - 0.0044L^2 + 0.000255L^3} [%] \tag{3.11}
\]
The value of $\Delta\zeta$ is the lower criteria for critical damping that is defined in Eurocode, SS-EN 1991-2:6.4.6.4(4). The total damping that a designer can use is obtained by Eq. (3.12). The damping provided by $\Delta\zeta$ is visualised in Figure 3.6.

$$\zeta_{total} = \zeta + \Delta\zeta$$  \hspace{1cm} (3.12)

Figure 3.6: Additional damping from Eurocode. SS-EN 1991-2: Figure 6.15 [25]

To simulate the high-speed traffic Eurocode has set up the designer with the HSLM-train. It constitutes of two universal trains which in Eurocode is denoted HSLM-A and HSLM-B. In this case only HSLM-A will be used for dynamic analysis. HSLM-A consists of ten reference trains and its basis of dynamic train signatures. For simply supported structures, its performance will give the midspan upper bound acceleration [23]. Figure 3.7 describes the load model HSLM-A. Each set of reference train, A1-A10, consist of different coach length, number of coaches, bogie-distance and point force.

<table>
<thead>
<tr>
<th>Universal train</th>
<th>Number of coaches, N</th>
<th>Length coach, D [m]</th>
<th>Bogie distance, d [m]</th>
<th>Point force, P [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>18</td>
<td>18</td>
<td>2.0</td>
<td>170</td>
</tr>
<tr>
<td>A2</td>
<td>17</td>
<td>19</td>
<td>3.5</td>
<td>200</td>
</tr>
<tr>
<td>A3</td>
<td>16</td>
<td>20</td>
<td>2.0</td>
<td>180</td>
</tr>
<tr>
<td>A4</td>
<td>15</td>
<td>21</td>
<td>3.0</td>
<td>190</td>
</tr>
<tr>
<td>A5</td>
<td>14</td>
<td>22</td>
<td>2.0</td>
<td>170</td>
</tr>
<tr>
<td>A6</td>
<td>13</td>
<td>23</td>
<td>2.0</td>
<td>180</td>
</tr>
<tr>
<td>A7</td>
<td>13</td>
<td>24</td>
<td>2.0</td>
<td>190</td>
</tr>
<tr>
<td>A8</td>
<td>12</td>
<td>25</td>
<td>2.5</td>
<td>190</td>
</tr>
<tr>
<td>A9</td>
<td>11</td>
<td>26</td>
<td>2.0</td>
<td>210</td>
</tr>
<tr>
<td>A10</td>
<td>11</td>
<td>27</td>
<td>2.0</td>
<td>210</td>
</tr>
</tbody>
</table>
3.2. Dynamic design for high-speed trains

The highest speed that is chosen to analyse is stated in the Eurocode, SS-EN 1991-2:6.4.6.2(1), that it should be $1.2 \times \text{Maximum Design Speed}$. The factor 1.2 does not include a future increase of speed. Thus, Eurocode recommends the designer to decide an additional factor to increase the speed when performing the dynamic analysis. For dynamic analysis, Eurocode suggests that only one track is assumed to be subjected to loads, SS-EN 1991-26.4.6.1.2(3). The chosen track should be the one that gives the most unfavourable situation.

The train model that will be used in this analysis is the HSLM-A 1-10 with speed in the ranges of 100-300 km/h, Eurocode recommends the designer to choose a speed increment that is small enough to capture resonance peaks occurring at different speeds during the analysis. In this thesis, the speed increment has been chosen to $\Delta v = 5$ km/h. This means that for each train-set a minimum of 40 different speeds is going to be analysed.

When a dynamic analysis is performed, a DAF $\left( \varphi'_{\text{dyn}} \right)$ should be determined from the ratio between dynamic and static displacements according to Eq. (3.13), SS-EN 1991-2:6.4.6.5(3). The factor considers general dynamic behaviour and in this case, it will be determined from a finite element-solution.

$$ \varphi'_{\text{dyn}} = \max \left| \frac{y'_{\text{dyn}}}{y_{\text{stat}}} \right| - 1 $$  \hspace{1cm} (3.13)

For comparison reasons, an additional DAF is evaluated by determining the DAF as the ratio between dynamic and static moments calculated at the crown of the SSCB as presented in Eq. (3.14). The motivations to estimate an additional dynamic factor other than the one suggested by Eurocode is to see if the DAF can differentiate, depending on which entity it is determined.

$$ \varphi'_{\text{dyn,M}} = \max \left| \frac{M'_{\text{dyn}}}{M_{\text{stat}}} \right| - 1 $$  \hspace{1cm} (3.14)

For real trainload and for fatigue limit state, the DAFs are obtained by following the guideline provided in Appendix C in SS-EN 1991-2. For carefully maintained track, the formulation is according to Eq. (3.15). The DAF according to Eq. (3.15) is applied on the static response. If dynamic response, i.e. dynamic stress signal, is available, the factor $\varphi'_{\text{dyn}}$ is disregarded in Eq. (3.15).

$$ 1 + \varphi = 1 + \varphi'_{\text{dyn}} + 0.5\varphi'' $$  \hspace{1cm} (3.15)

The factor $\varphi''$ is applied to take into account imperfections regarding the track. The definition from Eurocode is presented in Eq. (3.16). In the case for the fatigue analysis for SSCB, a carefully maintained track is assumed. The design value from the dynamic analysis is obtained by using Eq. (3.17), SS-EN1991-2:6.4.6.5(3).
\[ \phi^* = \frac{\alpha}{100} \left[ 56e^{\left(\frac{L_d}{10}\right)^2} + 50 \left( \frac{L_d}{80} - 1 \right) e^{\left(\frac{L_d}{20}\right)^2} \right] \geq 0 \]  
\[
\alpha = \begin{cases} 
\frac{v}{22} & v \leq 22 \frac{m}{s} \\
1 & v > 22 \frac{m}{s} 
\end{cases}
\] (3.16)

\[
\left(1 + \phi_{dyw} + 0.5\phi^*\right) \cdot \begin{cases} 
HSLM_L \quad \text{or} \\
RT 
\end{cases}
\] (3.17)

The final design value for the bridge should be the most unfavourable between Eq. (3.6) and Eq. (3.17) and the requirement for accelerations stated in SS-EN 1990:A2.4.4.2.1(4)P should be fulfilled.

### 3.2.2 Fatigue Limit Analysis

There is also a need of analysing FLS for SSCB since fatigue loading may initiate crack propagations, which can lead to rupture. Fatigue rupture is dependent of the stress variation and the number of cycles these stress variations are occurring. The cracks is usually initiated in locations where stress concentrations are occurring, such as at the crest or valley of the corrugation or at bolt holes. However, cracks may also occur at free surfaces because of material deviations in the microstructure of the steel plate. Thus, analysis for FLS should be performed in several locations that may suffer from stress concentrations.

Two methods are available when analysing FLS in general, Lambda method\(^8\) and Palmgren-Miner rule. The methods is provided in Eurocode and it is up to the designer choose which method to use. In general for FLS, TRVK Bro: B.3.2.1.4(j) clause decides that if a mixed traffic is occurring at the railway-line under design the designer does only need to design for FLS up to Maximum Design Speed set for the railway line according to SS-EN 1991-2: 6.9.

When FLS is analysed with Palmgren-Miner rule, the stress range Eurocode relates to could be described as in Figure 3.8. If it can be determined that the stress range is below the fatigue criteria, fatigue should not be a problem for the studied case, i.e. the

---

\(^8\) Presented in Chapter 3.1.
cumulative damage is estimated to zero. For the case in this thesis, the fatigue will be studied by using the Palmgren-Miner rule to calculate a stress collective and then determine the cumulative damage for each stress range.

The choice of analysing the dynamic stress range with Palmgren-Miner rule is since SS-EN 1991-2:6.4.6.6 [25] states that for dynamic analysis regarding FLS should consider the additional free vibrations, the amplitude of the stress range occurring at resonance from moving loads and additional stress cycles caused by dynamic loading at resonance. Moreover, a series of speeds up to the highest nominal speed, i.e. MDS, should be analysed to find the stress collective that estimates the largest cumulative damage from the dynamic stress-time history. If the dynamic stress-time history is determined from an FE-model, the stress signal should be amplified according to Eq. (3.18). The value of $\Delta \sigma_i$ should consider the whole stress signal from one passage of the applied trainload.

$$
\gamma_{fy} \Delta \sigma_i = \gamma_{fy} \Delta \sigma_i \cdot (1 + \phi) = \gamma_{fy} \Delta \sigma_i \cdot (1 + 0.5 \phi^*)
$$

(3.18)

In addition, the case when using HSLM-trains, an estimation of the future train traffic should be performed. The Palmgren-Miner rule for cumulative damage is defined as in Eq. (3.19). The total lifetime of the bridge can be estimated by calculating the total cumulative damage for a whole year and then use Eq. (3.20). Moreover, it is important to state which category number that is used when analysing the stresses with the Palmgren-miner rule according to Appendix A in SS-EN 1993-1-9:2005. In this thesis, the category number has been set to $\Delta \sigma_c=90$ MPa when performing the analysis with Palmgren-Miner rule for the bolted joint at the crown. The assumption of analysing the Palmgren-Miner rule with $\Delta \sigma_c=90$ MPa is a suggestion that not is available in the current handbook of report 112 nor TRVK Bro.

$$
D_j = \sum_i^N \frac{n_{E_j}}{N_{\text{eq}}}
$$

(3.19)

$$
Lifetime = \frac{1}{D_{\text{d,year}}}
$$

(3.20)
3.2.3 Swedish design regulations

Trafikverket has developed two national documents with general guidelines and design guidelines for bridge design that is conducted in Sweden. They are presented with their full name, abbreviated name and function. The abbreviated name will be used in the rest of the thesis. Both of the documents are only available in Swedish.

<table>
<thead>
<tr>
<th>Document full name</th>
<th>Abbreviated name</th>
<th>Function</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trafikverkets Krav Bro (TRV 2011:085)</td>
<td>TRVK Bro</td>
<td>Design guidelines</td>
</tr>
<tr>
<td>Trafikverkets Råd Bro (TRV 2011:086)</td>
<td>TRVR Bro</td>
<td>General guidelines</td>
</tr>
</tbody>
</table>

The documents replace the earlier used design documents TR Bro (VV 2009:028, BVH 1583.10) and TK Bro (VV 2009:27, BVS1583.10). The function of the documents is to provide the designer with general guidelines and design guidelines relevant for Sweden and thus over-rules the clauses provided in Eurocode. Both TRVR Bro [26] and TRVK Bro [27] contain guidelines for SSCB-structures regarding their design procedure, but nothing regarding dynamic analysis or high-speed traffic for SSCB. The parameters studied will follow the guidelines stated in TRVK- and TRVR Bro.

In general, the documents refer to the design handbook developed at The Royal Institute of Technology, ‘Design of soil-steel composite bridges’ – Report 112. Restrictions in the documents of the design lifetime for the SSCB-structures are set to 40 to 80 years depending on materials used to manufacture the pipe in use. As in this thesis a steel pipe is analysed, the design lifetime is thus set to 80 years.

TRVK Bro

For bolted joints, a category number $\Delta \sigma_c = 71$ MPa is allowed and over-rules what is said in report 112 for FLS according to TRVK Bro:J.3.2.2. In serviceability limit state, the document refers to SS-EN 1993-2: 7.3(1) to regulate normal- and shear-stresses. The criterion is valid for SSCB up to 5 m span length, and then further analysis is needed.

It can be noted that the predecessor document to TK Bro, BVS 583.10 [28], contains a suggested determinant length for SSCB-structures. This suggestion is not available in Eurocode. The suggestion is also given in report 112.

Soil-Steel Composite Bridges: $L_g = 2D$

TRVR Bro

A parameter for SSCB is the height of soil cover depth above the crown. For the static and dynamic reasons the parameter is important to estimate correctly. According to TRVR Bro:J.2.4, minimum soil cover depth should be 1.1 m consisting of 0.5 m of ballast and 0.6 m of soil cover. This is assumed to ensure stability requirements for the
SSCB if a future change of ballast will take place. As the SSCB structural system relies on the surrounding Young’s modulus of the engineered backfill it is also important to define this correctly. By following the documents clause, TRVR BroJ.3.2.1, it is regulated depending on which method is used that is defined in report 112, method A and method B are available. The maximum design value that a designer is allowed to use for a tangent soil modulus by using method A can be estimated to 10 MPa, which assumes long-term effects, i.e. static load cases. If method B is used, a higher Young’s modulus can be obtained. A deformation criteria for serviceability limit state is said to be needed for horizontal and vertical deformations based on the stress distribution in the pipe during the backfilling-process, the criteria is left for the designer to be determined.

3.2.4 Finite Element Procedure

The finite element procedure will be limited to equivalent 2D-models. Thus, some of the deformations occurring along the pipe axis, i.e. two directional bending in 3D action and twisting mode shapes, will not visible. However, the main difference is how the load is applied. This affects load distribution that results in differences for calculated deformations, stresses etc. Thus, generalizing a 3D real case scenario to a 2D-model gives secondary faults that cannot be controlled if the results that can be seen are not known how to be evaluated, i.e. few earlier cases, or if analytical results are not at hand or field measurements.

The chosen finite element program Brigade/PLUS is a program that is based on the ABAQUS solver. Thus, all information in the ABAQUS manuals is also applicable when using Brigade/PLUS. Brigade/PLUS contains information about Swedish design codes and thus it contains the HSLM-A pre-programmed and ready for the designer to use in the analysis.

A replica of the Märsta Bridge is modelled as a reference model and briefly analysed of generated results and compared to field measurements of the Märsta Bridge, result from the analysis will be presented with response from; displacements, strains and accelerations for the X52-train. This will make it able to calibrate the model until it provides results within reasonable error margins. When an accurate model is obtained for the Märsta Bridge, the model is rebuilt to fit the dynamic analysis for Björnbo Bridge. Thus, it will be stated if Björnbo Bridge will be able to withstand dynamic loading situations based on dimensions obtained from static load cases and what result that can be expected.

When a functional structure is determined for both static and dynamic load cases, a parametric study will be conducted. The parametric study will be confined to a set of parameters that are pre-determined from the literature review. The result that is analysed and compared is confined to accelerations, mode shapes with associated frequencies, displacements, moments and stresses. The results are extracted from nodal points and elements of interest along the circumference of the pipe and the level of ballast.

When design is performed according to report 112 the stiffness of the structure is calculated as a combined factor of both engineered backfill and SSCB. Whereas the Young’s modulus is estimated from the surrounding soil parameters for the engineered
backfill such as: stress levels, modulus ratio, stress exponent etc. In comparison to the FEM-procedure, the choice of Young’s modulus can be adjusted separately without adjusting the surrounding soil. Which make that the surrounding parameters does not affect the choice of Young’s modulus when performing FE-analysis for SSCB.

As also been presented is that the load model in report 112 constitutes of an equivalent static line load for the LM71-train. Compared to the trainload in the FE-analysis, the load consists of single moving point loads representing the axles of the moving train. Thus, the repetitive action of many axles passing the bridge is modelled which can cause resonance behaviour. Moreover, the interaction between train and bridge is not considered in this analysis since it would involve more detailed modelling of the spring-action from the train bogies.

When performing the FE-analysis the computational routine of modal analysis is chosen. Modal analyses are generally applied for structural dynamic problems by analysing separate modes and then use superposition techniques to obtain the result. Modal analysis is in general a fast numerical procedure, depending on how many modes that are analysed. However, the result from modal analysis can differentiate. This depend on how many modes that are considered in the analysis.

The damping in the FE-model is assigned with composite damping, for the case of continuum soil elements, the damping is assumed approximately to 2.0-2.5%.

Composite damping in ABAQUS manuals is described as a mean to calculate a model average critical damping with the material density as the weight factor. This means that Brigade/PLUS allows that a damping factor is defined for each material in the model, the factors are later combined into a damping factor for each mode as weighted averages of the mass matrix associated with each material as presented in Eq. (3.21), where there are no sum over sub-index $\alpha$. Thus, it provides mode-based dynamics, i.e. diagonal entries in the modal system of equations. This means that this approach of defining damping only allows for linear dynamic analysis.

$$\xi_\alpha = \frac{1}{m_\alpha} \phi_\alpha^M \left( \sum_m \xi_m^M M_m^M \right) \phi_\alpha^N$$  \hspace{1cm} (3.21)

The factor $m_\alpha$ is the generalized mass associated with mode $\alpha$, and in ABAQUS manuals it is defined as Eq. (3.22), no summation over sub-index $\alpha$.

$$m_\alpha = \phi_\alpha^M M^MN \phi_\alpha^N$$  \hspace{1cm} (3.22)

Brigade/PLUS uses the information from the designers input in damping properties of the materials to calculate damping coefficients $\xi_\alpha$ in the Eigen frequency extraction step from damping factors $\xi_m$, i.e. the factors defined by the designer in the material definitions. Composite modal damping is only available to use by specifying the number of modes the designer wants to account for in the FE-analysis. Thus, it is not only sufficient to specify a frequency range in the FE-analysis.
3.3 Model calibration of the Märska Bridge

Field Measurements

*The section about field measurements is extracted from chapter 2.2 in [34].*

The bridge was instrumented with a total of 12 strain gauges, 2 displacement transducers and 8 accelerometers. Data was collected using a HBM MGC-Plus A/D converter with 20 bit resolution. The sample frequency was set to 800 Hz and an analogue Bessel Low-Pass filter with a cut-off frequency of 400 Hz was employed to avoid aliasing. The instrumentation is presented in Figure 3.9 to Figure 3.11. Accelerometers are denoted a1-a8, displacement transducers d1-d2 and strain gauges e1-e12. Most gauges were positioned directly under the track U1, Figure 3.9. To gain further information on the load distribution however, d1 and e9-e12 were instrumented closer to track N1. Both displacement transducers measured the vertical crown deflection. Accelerometers were positioned both on the arch intrados and on the railway embankment, a3-a5 were positioned between the sleepers of track U1, both 20m before and after the culvert as well as directly over the crown.

![Figure 3.9](image)

*Figure 3.9: Instrumentation, Cross-section at the crown centre line.*

![Figure 3.10](image)

*Figure 3.10: Instrumentation, Section A-A, track U1.*

After 29 train passages, accelerometer a4-a6 was moved to positions indicated with *. Accelerometer a3-a5 was used to compare the response on the sleepers and the adjacent ballast, Figure 3.11a, a6 was used to determine any acceleration of the ground inside the culvert, directly above gauge d1, Figure 3.10. All strain gauges measured the longitudinal strain at the arch intrados, at each location both on the crest and on the
valley of the corrugation, Figure 3.11b. This facilitates estimation of the proportion of bending moment and axial force in the arch.

![Figure 3.11: Instrumentation, a) plane view of track U1, b) detail of the corrugated steel sheet.](image)

**Finite Element Model – Brigade/PLUS**

The first geometric model in Brigade/PLUS is created as a replica of the Märsta Bridge in a 2D-model with a longitudinal length of 50m with a planar rectangular surface that is symbolising the surrounding soil of the pipe. The height of the discretized soil profile is set to 10m, consisting of 0.5m thick ballast-layer and 9.5m thick engineered backfill layer.

The plain strain thickness in the z-direction is determined by assuming a load distribution of 2:1, starting from the bottom edge of the concrete sleepers and a start assumption made from drawings that the ballast layer is 4.5m wide in the z-direction (not visible in the FE-model).

Boundary conditions are applied at the edges of the soil profile with deformations locked in X- and Y-direction. Special attention is also given to the material definitions at the edges with ‘Silent boundaries’. The 50m long model is presented in Figure 3.12.

![Figure 3.12: 2D-model with boundaries extended](image)

9 A silent boundary is given high damping properties to delimit the reflecting waves from returning into the structure, which would disturb the results obtained in the FE-simulation.
The pipe is created with a single wire and assigned with a generalized profile that is given cross-sectional properties as calculated in a separate sub-model for one corrugation. This approach of modelling makes Brigade/PLUS approximately estimate the total stiffness for a whole pipe since the stiffness of one corrugation is added on along the direction of the wire which is symbolising the pipe. The wire is tied together with the surrounding soil with a fictitious constraint, allowing no friction to take place between the soil and the pipe.

In Brigade/PLUS, it is possible to calculate cross-sectional data for general profiles that may be used in some cases, in ABAQUS manuals referred to as *meshed beam cross-section* the general procedure can be found.

For this case, special type of elements is needed to be used, called warping elements. They are presented in *Table 3.3* and each element is presented in *Figure 3.13*. The warping elements consists of three active DOFs for triangular elements and four active DOFs for quadrilateral elements that describes the out-of-plane warping function and gives results for moment of inertia in X-, Y-direction and the product of moment of inertia (XY-direction), the torsional constant (J) and the area (A) for the created FEM-model. The values from the sub-model can later be used in the main FE-model representing Märsta Bridge. A single corrugation profile type 150x50, presented in *Figure 3.14*, for the Märsta Bridge is modelled to calculate cross-sectional properties with the use of the quadrilateral elements.

*Table 3.3:* Warping elements in Brigade/PLUS.

<table>
<thead>
<tr>
<th>Name</th>
<th>Element type</th>
<th>Active DOFs</th>
<th>Integration output (x)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WARP2D3</td>
<td>Triangular</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>WARP2D4</td>
<td>Quadrilateral</td>
<td>4</td>
<td>4</td>
</tr>
</tbody>
</table>

*Figure 3.13:* Warping elements.

*Figure 3.14:* Corrugation sub-model in Brigade/PLUS, Märsta Bridge. a), Single material definition, no boundary condition. b), Meshed region
The mesh for the corrugation is chosen very fine, a mesh of 1.0 mm is assigned to the region that is defining the corrugation since the FEM-model is quite small.

The sub-model of the corrugation gave the following cross-section properties presented in Table 3.4 that is used in the main model. Moreover, the obtained values for the cross-section in Brigade/PLUS is compared to obtained values for cross-sectional properties in a similar model created by supervisor Tech. Dr. Andreas Andersson in the FE-program SOLVIA.

Table 3.4: Cross-sectional properties 150x50 corrugation for Märsta Bridge.

<table>
<thead>
<tr>
<th>Young's modulus (E (GPa))</th>
<th>Area (A (m²))</th>
<th>Moment of inertia (Ixx (m⁴))</th>
<th>Moment of inertia (Iyy (m⁴))</th>
<th>Moment of inertia (Ixy (m⁴))</th>
<th>Torsional constant (J (m⁴))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brigade/PLUS</td>
<td>210</td>
<td>3.414·10⁻⁷</td>
<td>2.086·10⁻⁵</td>
<td>6.398·10⁻⁷</td>
<td>1.041·10⁻⁸</td>
</tr>
<tr>
<td>SOLVIA</td>
<td>210</td>
<td>3.415·10⁻⁷</td>
<td>2.088·10⁻⁵</td>
<td>6.430·10⁻⁷</td>
<td>1.047·10⁻⁸</td>
</tr>
</tbody>
</table>

The railway track is created with a wire that is assigned with a quadratic profile with an equivalent stiffness of a UIC60-railway profile by assuming the same moment of inertia, $I_{xx,UIC60} = 3038 cm^4$, and mass, $m_{UIC60} = 60.21 kg/m$, as for UIC60-profile, see derivation A in Eq. (3.23)-(3.24).

**Derivation A**

By setting the moment of inertia for the UIC60-profile ($I_{xx,UIC60}$) equal to the expression in Eq. (3.23), the height ($a$) for the quadratic profile is obtained as:

$$I_{y,z} = \frac{a^4}{12} \rightarrow a = \sqrt[4]{\frac{I_{xx,UIC60} \cdot 12}{I_{y,z}}} = 13.8 cm = 0.138 m \quad (3.23)$$

$$\rho_{eq} = \frac{m_{UIC60} \cdot L_{model}}{V} = 3161 kg/m^3 \quad (3.24)$$

An equivalent density for the rectangular profile is obtained by now being able to calculate the volume of the quadratic profile and using the known weight of the UIC60-
profile as presented in Eq. (3.24). Which gives a quadratic cross-section with the following properties as presented in Table 3.5\textsuperscript{10}.

**Table 3.5:** Sectional properties of discretized railway beam.

<table>
<thead>
<tr>
<th>Young's modulus ( E ) (GPa)</th>
<th>Density ( \rho_{eq} ) (kg/m(^3))</th>
<th>Height ( a ) (m)</th>
<th>Moment of inertia ( I_{y,z} ) (cm(^4))</th>
<th>Poisson ratio ( \nu ) (-)</th>
<th>Damping ( \zeta ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>196</td>
<td>3161</td>
<td>0.138</td>
<td>3022</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

The railway track is tied together with the concrete sleepers with a fictitious constraint to create a load transferring from the loads applied on the rail via concrete sleepers and further down in the soil profile. In Figure 3.15, the stress distribution from a moving point load of 100 kN moving over the 50m long model is presented. The influence line for the vertical displacement for the Märsta Bridge is also presented. The reduced model is presented in Figure 3.16. The material definitions of the model are presented in Table 3.6, which is taken as characteristic design values. From the information obtained from the stress distribution, the models length is reduced to 26m to minimize the computational effort that is needed from the computer.

The soil cover depth, estimated from top corrugation to bottom edge of railway beam, in the FE-model is set to a total of 1.7m that was estimated during the collection of measurements. During the procedure of reducing the model, the mesh is set to a mesh size approximately to 0.25m, and near the circumference of the pipe, the mesh is set 0.2-0.1m to be sure of obtaining an accurate result. Nodal result points for Märsta Bridge are presented in Figure 3.17 and are following the same notation as was for the field measurements.

**Table 3.6:** Material definitions for replica model of Märsta Bridge.

<table>
<thead>
<tr>
<th>Parts</th>
<th>Plain strain thickness (m)</th>
<th>Young's modulus ( E ) (GPa)</th>
<th>Density ( \rho_{eq} ) (kg/m(^3))</th>
<th>Damping ( \zeta ) (%)</th>
<th>Poisson ratio ( \nu ) (-)</th>
<th>Color code</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete sleepers</td>
<td>3.0</td>
<td>35</td>
<td>2000</td>
<td>0</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>SSCB-profile</td>
<td>-</td>
<td>210</td>
<td>7800</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Ballast</td>
<td>4.5</td>
<td>0.300</td>
<td>1700/2100</td>
<td>0</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>Engineered backfill</td>
<td>5.7</td>
<td>0.220</td>
<td>1850</td>
<td>2.5</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>Silent boundary ballast</td>
<td>4.5</td>
<td>0.300</td>
<td>1700</td>
<td>100</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>Silent boundary backfill</td>
<td>5.7</td>
<td>0.220</td>
<td>1850</td>
<td>100</td>
<td>0.2</td>
<td></td>
</tr>
</tbody>
</table>

\textsuperscript{10} Note that this is the discretized properties for one single beam since the FE-analysis is confined to 2D.
Figure 3.15: Stress distribution and influence line for the crown for Märsta Bridge.

Figure 3.16: Reduced model of Märsta Bridge and nodal result points.

Figure 3.17: Nodal result points bridge profile in Brigade/PLUS, same notations as measurements from Märsta Bridge.
Convergence studies

For the railway-track and Märsta Bridge profile, Beam elements following Euler-Bernoulli theory is used, listed in the ABAQUS manual as:

- B23 – 2-node cubic beam in a plane

The convergence analysis for the beam elements was performed in a separate 2D sub-model by analysing a simply supported beam that is compared to results from another FE-program, SOLVIA. The results from SOLVIA are provided from supervisor Tech. Dr. Andreas Andersson. The analysis in the sub-model presented converged results with a mesh-size of the beam element to 0.5m. The comparison of results between SOLVIA and Brigade/PLUS are presented in Table 3.7. The comparison was mainly in finding a convergence for frequencies and accelerations to be sure of obtaining a correct result from a dynamic response point of view.

Table 3.7: Sub-model of simply supported beam.

<table>
<thead>
<tr>
<th>Entity</th>
<th>SOLVIA</th>
<th>Brigade/Plus</th>
</tr>
</thead>
<tbody>
<tr>
<td>First frequency [Hz]</td>
<td>2.3</td>
<td>2.3</td>
</tr>
<tr>
<td>Second frequency [Hz]</td>
<td>9.1</td>
<td>9.1</td>
</tr>
<tr>
<td>Third frequency [Hz]</td>
<td>20.3</td>
<td>20.4</td>
</tr>
<tr>
<td>Acceleration [m/s²]</td>
<td>6.6</td>
<td>6.4</td>
</tr>
<tr>
<td>Static displacement [mm]</td>
<td>11.3</td>
<td>11.7</td>
</tr>
<tr>
<td>Static moment [MNm]</td>
<td>-7.8</td>
<td>-7.72</td>
</tr>
</tbody>
</table>

To be sure of obtaining reasonable results from FE-simulations, a convergence analysis of the mesh is performed. The convergence studies for the calibration model are performed for three kinds of shell elements, listed below:

- CPE3 – 3-node linear, Plain strain triangle
- CPE8 – 8 node biquadratic, Plain strain quadrilateral (full integration)
- CPE8R – 8-node biquadratic, Plain strain quadrilateral (reduced integration)

The mesh convergence is checked for the entities: Displacement, axial stress, axial force and bending moment about the local X-axis. The goal with a convergence analysis is to find the mesh-size that will converge to the exact solution within the mathematic model as the mesh is indefinitely refined. This is performed by neglecting the errors due to finite computer arithmetic [31]. Still, there is a give and take in this procedure since the designer does not want to refine the mesh too much and spending a lot of unnecessary CPU-power. Thus, a well-suited approach could be to find the mesh-size that converges to the right result and in certain parts of the FE-model make the mesh-size smaller than necessary to in certain parts get higher precision in the calculated result.

The convergence studies in this analysis are performed by studying the result in a single node/element in the crown of the pipe. The load that is subjected in the model consists of a single moving point load of 100 kN, the convergence for the chosen entities are presented in Figure 3.18
Figure 3.18: Convergence studies Märsta Bridge.

As can be seen from Figure 3.18, convergence is reached when the mesh-size is set to 0.25m near the circumference of the pipe for each type of element. As can be seen also is that the 8-noded elements has a higher converge rate, i.e. they converge faster to the correct result.

The difference between CPE8 and CPE8R is the number of integration points that are performed within the element. Full integration is defined as integrating all stiffness parameters that belongs to the element, which provides a very stiff element. By using reduced integration, the computational time is reduced and all the stiffness coefficients is not integrated, which gives less stiffness to the element. If the computational time is minimized it is favourable for nonlinear/dynamic FE-analysis, and as in this case a dynamic analysis is performed, the final element choice is set to CPE8R with a mesh-size equal to 0.25m.

Since finer precision in calculated result is wanted around the circumference of the SSCB, the mesh is refined to the boundaries where the soil and pipe meet each other. Thus, the final mesh of the main FE-model is presented in Figure 3.19. A summarized table of the mesh properties is given in Table 3.8.
Table 3.8: Mesh properties of main FE-model of Märsta Bridge.

<table>
<thead>
<tr>
<th>Part</th>
<th>Element type</th>
<th>Mesh size (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Railway track</td>
<td>2-node cubic beam in plane (B23)</td>
<td>0.5</td>
</tr>
<tr>
<td>SSCB Profile</td>
<td>2-node cubic beam in plane (B23)</td>
<td>0.1</td>
</tr>
<tr>
<td>Ballast</td>
<td>8-node biquadratic, plain strain quadrilateral with reduced integration (CPE8R)</td>
<td>0.25-0.1</td>
</tr>
<tr>
<td>Engineered backfill</td>
<td>8-node biquadratic, plain strain quadrilateral with reduced integration (CPE8R)</td>
<td>0.25-0.1</td>
</tr>
<tr>
<td>Silent boundary Ballast</td>
<td>8-node biquadratic, plain strain quadrilateral with reduced integration (CPE8R)</td>
<td>0.25</td>
</tr>
<tr>
<td>Silent Boundary Backfill</td>
<td>8-node biquadratic, plain strain quadrilateral with reduced integration (CPE8R)</td>
<td>0.25</td>
</tr>
<tr>
<td>Concrete sleeper</td>
<td>8-node biquadratic, plain strain quadrilateral with reduced integration (CPE8R)</td>
<td>0.1</td>
</tr>
</tbody>
</table>

Figure 3.19: Mesh appearance in main FE-model of Märsta Bridge.

With the mesh of the model decided, it is of importance to estimate the correct response for accelerations by assuming a small enough time step. In the same manner, as for the mesh-convergence analysis, the approach where the time step is indefinitely refined is used. The analysis for different time steps is presented in Figure 3.20, where it is seen that when the time step is refined to 2 ms the acceleration has reached a converged value. However, since there are some discrepancies in the estimated accelerations when the time step is smaller than 1 ms, a choice is made to set the final time step to 1 ms to be sure of reaching a converged value. The analysis was performed for a single moving point load of 100 kN with CPE8R-elements.
CHAPTER 3. DESIGN OF SOIL-STEEL COMPOSITE BRIDGES

Figure 3.20: Time step convergence analysis.

Calibration process – X52-train

The field measurements that are available in this thesis for the Märsta Bridge are measurements from the passing of the X52-train. The X52-train is a commuter train with an axle-load of 185 kN, a total length of 48.7m between the outer axles of the two wagons. A detailed section of the X52-train is presented in Figure 3.21. The X52-train travels with a speed approximately between 180-200 km/h, during the measurements the speed of the train is estimated to about 171 km/h by taking the total length of the train and divide it with the time difference between the passage of the first and last bogie. Thus, in the FE-analysis in Brigade/PLUS it has been assumed that the train travels with a speed of 180 km/h. The calibration process is performed for displacement, strains and accelerations. This process of calibration for displacements and strains can be described as by changing the stiffness of the different discretized parameters for; soil profile, railway beam, SSCB-profile etc. The calibration of the accelerations is described by changing the mass properties, i.e. densities, of the discretized parameters. Thus, this procedure gave the final material properties as earlier presented in Table 3.6.

Figure 3.21: X52-train section, axle distance in metre.

Consequently as a measure of agreement between field measurement and estimated displacements, strains and accelerations in FE-simulations, the following method has been utilized as presented in Eq. (3.25), where so is possible. This gives the difference between values obtained in FE-solution and field measurements in ratios.

\[
\text{Difference} = \max \left| \frac{x_i}{y_i} \right|
\]  (3.25)
where:

\( x_i \) - Is the maximum value of; displacements, strains or accelerations at an arbitrarily chosen point of the estimated signal from FE-simulations

\( y_i \) - Is the maximum value from field measurement of; displacements, strains or accelerations at an arbitrarily chosen point of the measured signal

The comparison between FE-simulations and the field measurements are presented in Table 3.9\(^\text{11}\) for crown displacement and accelerations while the results from simulated strains are presented in Table 3.10.

Table 3.9: Comparison between field measurement and FEM 2D results for displacements and acceleration.

<table>
<thead>
<tr>
<th>Measure point</th>
<th>d1</th>
<th>a1</th>
<th>a2</th>
<th>a3</th>
<th>a4</th>
<th>a5</th>
<th>a6</th>
</tr>
</thead>
<tbody>
<tr>
<td>FEM 2D</td>
<td>0.39</td>
<td>0.21</td>
<td>0.20</td>
<td>0.19</td>
<td>0.17</td>
<td>0.12</td>
<td>0.07</td>
</tr>
<tr>
<td>Field measurement</td>
<td>0.38</td>
<td>0.80</td>
<td>0.33</td>
<td>1.33</td>
<td>-</td>
<td>1.31</td>
<td>0.12</td>
</tr>
<tr>
<td>Difference</td>
<td>1.0</td>
<td>0.3</td>
<td>0.6</td>
<td>0.1</td>
<td>-</td>
<td>0.1</td>
<td>0.6</td>
</tr>
</tbody>
</table>

Table 3.10: Comparison between field measurements and FEM 2D results for strains.

<table>
<thead>
<tr>
<th>Measure point</th>
<th>e1</th>
<th>e2</th>
<th>e3</th>
<th>e4</th>
<th>e5</th>
<th>e6</th>
<th>e7</th>
<th>e8</th>
</tr>
</thead>
<tbody>
<tr>
<td>FEM 2D</td>
<td>45.8</td>
<td>42.1</td>
<td>29.7</td>
<td>24.2</td>
<td>17.3</td>
<td>10.3</td>
<td>26.3</td>
<td>22.4</td>
</tr>
<tr>
<td>Field measurement</td>
<td>13.6</td>
<td>11.9</td>
<td>27.0</td>
<td>18.3</td>
<td>18.5</td>
<td>24.9</td>
<td>33.3</td>
<td>15.7</td>
</tr>
<tr>
<td>Difference</td>
<td>3.4</td>
<td>3.5</td>
<td>1.1</td>
<td>1.3</td>
<td>0.9</td>
<td>0.4</td>
<td>0.8</td>
<td>1.4</td>
</tr>
</tbody>
</table>

The obtained results from FEM simulations present that the estimated displacements agree well with the field measurements. While the accelerations that is simulated from FE-solution is greatly underestimated compared to the measured accelerations from the field. The strains that are simulated from FE-solution agree well in some of the measured points, while some of the points suffer from over-/under-estimated strains.

The difference in results for accelerations is believed to be not being able to model the correct mass-components since the acceleration-levels can be connected with the modelled mass of a structure. Moreover, the stiffness of the engineered backfill with a dynamic modulus may give that the correct acceleration-levels cannot be estimated correctly.

Moreover, by analysing the contour plot of the max envelope from vertical accelerations, Figure 3.22, from one passage of the X52-train, it shows that a larger

---

\(^{11}\) Note, in Table 3.9 that accelerometer a4 was later found out that it was broken during the measurements and thus, only calculated values from FE-simulations are available for this measurement point.
acceleration may have been observed in the FEM-simulation if the node to analyse would have been chosen to the left quarter point of the Märsta Bridge. However, the scale presented connected to the contour plot reveals that the acceleration that would have been observed would have still been smaller than the measured acceleration at the crown.

![Contour plot of vertical accelerations from one passage of the X52-train.](image)

The strains that are simulated with the 2D-equivalent model agree well in some points. However, the reason for not being able to estimate correct strains in several locations may be the effect of assuming a dynamic soil modulus, which affects the load distribution by the depth in the soil. At the time of this thesis, there are no available methods on how to estimate a dynamic Young’s modulus that could be connected with the static Young’s modulus estimated in report 112. Moreover, another reason for not being able to estimate the correct strains may be simplifying the corrugated steel plate with a generalized profile.

During the calibration of the model, the discretized parameter that had largest effect was the Young’s modulus of the engineered backfill when analysing the displacements of the crown region. The discretized parameter for Young’s modulus for SSCB-profile or railway beam did not affect the resulting displacement in the same manner. When changing the stiffness of the engineered backfill, the distribution of the vertical acceleration also was affected. The discretized mass properties of the equivalent 2D-model had largest effect on the estimated vertical acceleration when changing the discretized mass of the engineered backfill. Since the discretized SSCB contributes with little mass in comparison to the discretized engineered backfill, the changing of its discretized mass did not affect the vertical acceleration.

Comparing the analysis performed in this thesis with Mellat [3], Mellat estimated a crown displacement of 0.49mm while the crown acceleration was estimated to 1.53m/s² for Märsta Bridge for the X52-train. This indicates that there can occur discrepancies in-between FE-models of simulated results.

### 3.4 HSLM-A2 analysis for Märsta Bridge – Dynamic analysis

**Damping/Mode-shape analysis/Steady-state analysis – Märsta Bridge**

By following recommendations in Eurocode to analyse the structure within a frequency range up to the highest of \([30, 1.5f_1, f_3]\) (Hz), where the chosen limit is 30 (Hz),
Brigade/PLUS estimates 10 modes. All modes are considered in the FE-analysis and the damping for each mode is presented in Table 3.11.

Table 3.11: Composite modal damping Märsta Bridge.

<table>
<thead>
<tr>
<th>Mode (No.)</th>
<th>Frequency (Hz)</th>
<th>Composite modal damping (%)</th>
<th>Mode (No.)</th>
<th>Frequency (Hz)</th>
<th>Composite modal damping (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>9.72</td>
<td>2.22</td>
<td>1</td>
<td>9.82</td>
<td>2.27</td>
</tr>
<tr>
<td>2</td>
<td>10.51</td>
<td>2.31</td>
<td>2</td>
<td>10.58</td>
<td>2.35</td>
</tr>
<tr>
<td>3</td>
<td>11.34</td>
<td>2.30</td>
<td>3</td>
<td>11.43</td>
<td>2.33</td>
</tr>
<tr>
<td>4</td>
<td>13.21</td>
<td>2.38</td>
<td>4</td>
<td>13.27</td>
<td>2.41</td>
</tr>
<tr>
<td>5</td>
<td>15.13</td>
<td>2.34</td>
<td>5</td>
<td>15.23</td>
<td>2.37</td>
</tr>
<tr>
<td>6</td>
<td>16.17</td>
<td>2.38</td>
<td>6</td>
<td>16.27</td>
<td>2.41</td>
</tr>
<tr>
<td>7</td>
<td>19.36</td>
<td>2.39</td>
<td>7</td>
<td>19.48</td>
<td>2.42</td>
</tr>
<tr>
<td>8</td>
<td>20.46</td>
<td>2.36</td>
<td>8</td>
<td>20.61</td>
<td>2.39</td>
</tr>
<tr>
<td>9</td>
<td>22.98</td>
<td>2.50</td>
<td>9</td>
<td>23.04</td>
<td>2.51</td>
</tr>
<tr>
<td>10</td>
<td>23.42</td>
<td>2.40</td>
<td>10</td>
<td>23.56</td>
<td>2.43</td>
</tr>
</tbody>
</table>

The mode shapes connected with each excited frequency connected with high-density ballast are presented in Figure 3.23. The mode shapes indicates that the surrounding soil is highly interactive with the modelled bridge profile. From the appearance of the mode shapes, it can be noted that some are active in the longitudinal direction and some are active in the vertical direction. However, the real case scenario, i.e. 3D-case, might resemble other types of modes with twisting action of the bridge profile.

![Mode shapes](image1)

Figure 3.23: Mode-shape analysis Märsta Bridge.

Regarding the estimation of the frequencies for these types of FE-models, it should be remembered that the frequencies could be estimated differently depending on the
CHAPTER 3. DESIGN OF SOIL-STEEL COMPOSITE BRIDGES

longitudinal size of the model for continuum elements. Therefore, an analysis should always be performed on estimated frequencies and mode shapes on a larger size model before making simplifications such as making the longitudinal length of the FE-model shorter.

The total damping in the FE-model is analysed with the Half Power Bandwidth method [32], where the damping is calculated as presented below in Eq. (3.26) and Figure 3.24 for an obtained signal with FE-analysis. The analyse could be useful since modelling the surrounding soil in the FE-model will add some damping in the form of radiation damping from the soil or numerical damping from the FE-model elements.

\[ \zeta = \frac{f_b - f_a}{f_b + f_a} \]  

(3.26)

![Figure 3.24: Half Power Bandwidth method.](image)

The half power bandwidth method can be used if the designer performs a steady-state analysis. Analyse can be of the displacements at the crown from a steady-state load with the same magnitude as an axle load from the train involved in the analysis.

To analyse the total damping in the structure that is provided from the surrounding soil, the Half Power Bandwidth method [32] is used by analysing a steady-state load applied above the crown on the railway track of 200 kN. From the steady-state response, it is possible to analyse resonance behaviour of the structure from applied load. In addition, it is possible to analyse the phase angle, which represents the delayed response of the structure from the applied load and resonance behaviour. The steady-state analysis and damping of the structure is presented in Figure 3.25.

As is seen is that the surrounding soil provides damping in the FE-model by an additional 2.5%, giving a total damping of 5.1% at the crown of the Märsta Bridge by analysing the first resonance peak. In addition, four peaks can be located for the crown of Märsta Bridge, which is indicating that the structure is in resonance with the applied load.
3.4. HSLM-A2 ANALYSIS FOR MÄRSTA BRIDGE – DYNAMIC ANALYSIS

The Märsta Bridge is analysed for the HSLM-A2 train set to be able to state whether Märsta Bridge can be subjected to high-speed trains. The analysis for the HSLM-A2 train is performed with the same material definitions as for the X52-train.

The response for displacements from the HSLM-A2 train is presented in Figure 3.26\textsuperscript{\textsuperscript{12}. The minimum envelope occurs at 300 km/h with the displacement 0.63 mm. The time history for the dynamic displacement is compared to the static displacement from a passage of the HSLM-A2 train, which indicates that there is a case of quasi-static displacements for dynamic analysis when analysing SSCB.

\textbf{HSLM-A2 analysis for Märsta Bridge}

The Märsta Bridge is analysed for the HSLM-A2 train set to be able to state whether Märsta Bridge can be subjected to high-speed trains. The analysis for the HSLM-A2 train is performed with the same material definitions as for the X52-train.

The response for displacements from the HSLM-A2 train is presented in Figure 3.26\textsuperscript{\textsuperscript{12}. The minimum envelope occurs at 300 km/h with the displacement 0.63 mm. The time history for the dynamic displacement is compared to the static displacement from a passage of the HSLM-A2 train, which indicates that there is a case of quasi-static displacements for dynamic analysis when analysing SSCB.

\textsuperscript{12} Note that the minimum envelope presented in Figure 3.26 is the deformation of main interest, which represent the downward deflection of the crown. In Figure 3.26 presented as the absolute value.
The calculated accelerations for the crown for Märsta Bridge are presented in Figure 3.27 for high-/low-density ballast. As can be seen in the presented envelope, the acceleration is below the limit stated in Eurocode, 3.5 m/s² for ballasted tracks. The maximum acceleration is occurring at 280 km/h for high-density ballast and is estimated to 1.26 m/s². The time history for when the train is passing the bridge at a speed of 280 km/h is presented for high-/low-density ballast.

![Figure 3.27a](image1)  
![Figure 3.27b](image2)

**Figure 3.27:** Acceleration response from HSLM-A2 for Märsta Bridge.

It can be noted that there are small differences for the estimated accelerations depending on which mass that is used for the ballast. In the time history for \( v = 280 \) km/h, the acceleration peak is located approximately at 0.9 s. The time where the maximum peak of acceleration is located is the time since the first point load entered the FE-model.

In addition, by analysing the contour plot of the maximum envelope for vertical accelerations in Figure 3.28 from one passage of the HSLM-A2-train. It is noticed that the maximum vertical acceleration is occurring in the crown region.

![Figure 3.28](image3)

**Figure 3.28:** Contour plot of vertical acceleration from one passage of HSLM-A2, Märsta Bridge.
In Table 3.12, Eq. (3.27) has been used and is compared to the frequencies obtained in the steady-state analysis for the Märsta Bridge to find possible velocities where the trainload may be in resonance with the crown. Since the HSLM-A2 trainload has been used during analysis, the bogie distance is set to $\lambda = 3.5$ m as presented in Figure 3.7.

$$v_i = n_i \lambda \quad \rightarrow \quad n_i = \frac{v_i}{\lambda}$$  

(3.27)

Table 3.12: Possible velocities where the HSLM-A2 trainload and the crown of Märsta Bridge may be in resonance.

<table>
<thead>
<tr>
<th>Resonance speed $v_i$, Figure 3.27 (km/h)</th>
<th>Resonance frequency $n_i$, Eq. (3.27) (Hz)</th>
<th>Frequency from steady-state analysis Figure 3.25 (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>125</td>
<td>9.9</td>
<td>9.8</td>
</tr>
<tr>
<td>150</td>
<td>11.9</td>
<td>13.3</td>
</tr>
<tr>
<td>230</td>
<td>18.3</td>
<td>16.3</td>
</tr>
<tr>
<td>280</td>
<td>22.2</td>
<td>20.7</td>
</tr>
</tbody>
</table>

There is noticed that the first estimated frequency for 125 km/h is almost estimated to the same value, which indicates that resonance may occur at the speed of 125 km/h for Märsta Bridge.

The largest moments in the FE-model from the applied load of the HSLM-A2 is presented in Figure 3.29. Small differences can be seen between high-/low-density ballast. Moreover, the largest moment is occurring at 275 km/h and the maximum positive moment is estimated to 6.3 Nm while the minimum negative moment is estimated to -15.6 Nm. The dynamic moment is compared to the static moment of a passing HSLM-A2 train.

Figure 3.29: Moments response from HSLM-A2 for Märsta Bridge.
DAFs for Märsta Bridge are determined by analysing the absolute value of the maximum dynamic displacement and the absolute value at an arbitrary point for static displacement, i.e. Eq. (3.13), same procedure is used for the moment, i.e. Eq. (3.14). The DAFs for each speed is presented in Figure 3.30.

![Dynamic Amplification Factors](image)

**Figure 3.30:** DAFs ($\varphi_{\text{dyn}}$) from displacement and moment for Märsta Bridge.

The DAF is calculated as recommended from Eurocode, i.e. for dynamic and static displacements, which gives that the largest DAF is estimated when the train passes the bridge at a speed of 300 km/h. The additional DAF that is analysed in this thesis, i.e. for dynamic and static moments, gives a larger DAF than the one suggested by Eurocode and is estimated when the train passes the bridge at a speed of 280 km/h. The DAF obtained in a dynamic analysis for higher speeds should be compared to DAF determined with Eq. (3.9) since the DAF determined with the equation can be estimated larger. Thus, the final DAF should be chosen as the largest value to use in design.

**Fatigue Limit State – Märsta Bridge**

DAFs are determined regarding FLS, which consider the fundamental frequency and the determinant length, SS-EN 1991-2 Appendix C takes into account the track irregularities, Eq. (3.16). The DAF is presented in Table 3.13. Moreover, since the dynamic time-history for stresses is available the DAF in Eq. (3.15) is determined without $\varphi_{\text{dyn}}$ and since the DAF is larger from the length of the influence line from Brigade/PLUS, the DAF equal to 1.15 is used in the FLS-analysis.

---

Note that the determinant length according to report 112 is calculated as a comparison to the influence length determined in FE-simulations. However, the analysis of the influence length in the FE-model is larger, approximately 16.5m, Figure 3.15.
To analyse Märsta Bridge for fatigue there is of interest to find the stress range that occur when the train is passing and being able to decide at which speed the maximum stress range is occurring. By analysing the envelope for stresses in Figure 3.31 it can be seen that the largest characteristic stress is occurring at 300 km/h and is estimated to 14.3 MPa. Thus, the stress-time history for 300 km/h is analysed with the Palmgren-Miner rule, presented in Figure 3.32. The stress-history is multiplied with the recommended DAF as recently presented and $\gamma_{fy} = 1.35$.

### Table 3.13: DAFs for FLS for Märsta Bridge.

<table>
<thead>
<tr>
<th>Brigade/PLUS</th>
<th>$L_0$ (m)</th>
<th>$1+0.5 \varphi''$ (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brigade/PLUS</td>
<td>7.5</td>
<td>1.146</td>
</tr>
<tr>
<td>Brigade/PLUS</td>
<td>16.5</td>
<td>1.153</td>
</tr>
</tbody>
</table>

**Figure 3.31:** Characteristic stress envelope for HSLM-A2 for Märsta Bridge.

**Figure 3.32:** Märsta Bridge analysed with the Palmgren-Miner rule. Stress signal amplified.
As can be seen from the presented stress collective in Figure 3.32 is that the dominating stress ranges is below 10 MPa and some stress ranges is estimated in-between 20-25 MPa. However, the cumulative damages from this stress collective for one passage of the HSLM-A2 train are estimated to be zero since the maximum stress is below the failure criteria. Thus, the Märsta Bridge has infinite lifetime regarding FLS.
Chapter 4

Case study of the Björnbo Bridge

The following chapter presents the case study of Björnbo Bridge. The Björnbo Bridge has been analysed with static design methods according to report 112 and a brief summary of the utilization rates and details about the cross-section is presented. An attempt is made to verify the Björnbo Bridge in a dynamic analysis with methods that are described in Eurocode documents. The results are here presented with comments from the dynamic analysis, chapter 4.3.

4.1 Properties of the bridge

Björnbo Bridge

Björnbo Bridge is located outside the town Skutskär. Skutskär is located in Sweden, approximately 160 km north of Stockholm and Björnbo Bridge is constructed in the railway section on the east coast railway between Skutskär-Furuvik, presented in Figure 4.1. The Björnbo Bridge is a future planned SSCB that will fulfil a pedestrian and cycle path with a high-speed railway passing above.

Figure 4.1: Location of Björnbo Bridge. Source: Google maps.
The railway line connected to this thesis is a part of the east-coast railway between Skutskär-Furuvik (km 99+962). The maximum design speed is going to be 250 km/h for high-speed trains and 200 km/h for conventional trains after the upgrade of the track has been carried out into a two-track railway. The signal system for the upgraded track is constructed for speeds up to 200 km/h. The maximum axle-load for passing trains is 25 tonnes, i.e. 250 kN. After measuring the amount of traffic during 2009 it was estimated that the east-coast railway at the current section had 94 passages per day, the forecast of the traffic 2020 is that approximately 147 passages per day is going to occur, see more detailed information of the mixed traffic in Table 4.1

Table 4.1: Current traffic and forecast traffic on the east-coast railway between Skutskär-Furuvik.

<table>
<thead>
<tr>
<th>Train-type</th>
<th>Train per day, 2009</th>
<th>Train per day, 2020</th>
</tr>
</thead>
<tbody>
<tr>
<td>High-speed train</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>Interregio-trains</td>
<td>18</td>
<td>26</td>
</tr>
<tr>
<td>Local trains</td>
<td>20</td>
<td>40</td>
</tr>
<tr>
<td>Night trains</td>
<td>7</td>
<td>8</td>
</tr>
<tr>
<td>Freight trains</td>
<td>29</td>
<td>43</td>
</tr>
<tr>
<td>Summation</td>
<td>94</td>
<td>147</td>
</tr>
</tbody>
</table>

General design assumptions

The case for Björnbo Bridge is that two railway tracks are going to be located on top of the railway embankment. The static design will however be performed for only one railway track since the dynamic analysis is performed with only one railway track. Thus, this will make the results obtained from the dynamic analysis more comparable to the static design. As can be seen in Figure 4.3, the SSCB foundation is in a small inclination hence giving different soil cover depth depending on which track that is basis for the static analysis. Thus, an analysis has been performed for the railway track with southbound traffic and northbound traffic to decide which of the tracks that gives the highest utilization rates for the SSCB.

The highest utilization rates from verifications according to report 112 were obtained for the track with traffic travelling northbound and thus, the presented values in the following chapter regard surrounding environment located at northbound railway track for the Björnbo Bridge.

The static design will solely be following the guideline provided from report 112. The corrosion of steel is neglected and hence no reduction of the thickness of the steel is considered. This simplification is made since counter-actions against corrosion can differentiate depending on which international design code that is basis for the design for SSCB. The characteristic values for the steel is presented in Table 4.2

---

14 Interregio-trains is a compliment to high-speed trains that will fulfil fast travels in shorter distances and high-speed trains support fast travels for longer distances.
4.1. Properties of the Bridge

Table 4.2: Characteristic values for steel.

<table>
<thead>
<tr>
<th>Description</th>
<th>Yield strength</th>
<th>Ultimate strength</th>
<th>Young’s modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Notation</td>
<td>$f_{yk}$</td>
<td>$f_{uk}$</td>
<td>$E_s$</td>
</tr>
<tr>
<td>Unit</td>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[GPa]</td>
</tr>
<tr>
<td>Design value</td>
<td>355</td>
<td>430</td>
<td>210</td>
</tr>
</tbody>
</table>

The bolts that are calculated for the Björnbo Bridge is calculated with a resulting strength class of 8.8 according to SS-EN 1993-1-8:3.1.1(3) [41] and dimension M20 according to material standardization document SS-EN ISO 898-1:2009. The characteristic values of the bolts are presented in Table 4.3

Table 4.3: Characteristic values for bolts.

<table>
<thead>
<tr>
<th>Description</th>
<th>Yield strength</th>
<th>Ultimate strength</th>
<th>Nominal area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Notation</td>
<td>$f_{yb}$</td>
<td>$f_{ub}$</td>
<td>$A_s$</td>
</tr>
<tr>
<td>Unit</td>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[mm$^2$]</td>
</tr>
<tr>
<td>Design value</td>
<td>640</td>
<td>800</td>
<td>245</td>
</tr>
</tbody>
</table>

At hand are preliminary drawings to find suggested profiles for the SSCB. The bridge is going to consist of a closed pipe-arch corrugated steel culvert with three radii, i.e. VT-profile. Its cross-section consists of a corrugation 200x55 mm. Viacon manufactures the SSCB of the Björnbo Bridge. Viacon has a set of standard profiles ready to be used for SSCB. However, for the case of Björnbo Bridge, a new profile has been created. The horizontal and vertical span is 2.528m and 2.394m respectively, excluding the height of corrugation. An elevation of the cross-section with the radii’s and corrugation is presented in Figure 4.2.

Figure 4.2: Profile for the Björnbo Bridge.

The cross-section at the centreline is presented in Figure 4.3 and an elevation of the structure is presented in Figure 4.4 with foundation height for the pipe and height of railway track at the crossing of the railway track with linear interpolation from measured point at km 99+960 and km 99+970, Figure 4.4. The railway track and railway sleepers is founded on ballast. A perspective drawing with the combination of the cross-section and elevation is presented in Figure 4.5 where the naked pipe is visible before backfilling and the finalized structure. The soil cover depth in this
analysis is assumed from crest of the corrugation to the bottom edge of the railway sleeper. This gives a soil cover depth of 1.57m.

Figure 4.3: Cross-section at centreline based on preliminary drawings of the crown for Björnbo Bridge. Note that preliminary dimensions are presented in this figure.

Figure 4.4: Elevation based on preliminary drawings for Björnbo Bridge. Note that preliminary dimensions are presented in this figure.
4.2 Static design

Dimensions Björnbo Bridge

The design of Björnbo Bridge is performed with the help of the design calculation presented in Appendix A. This resulted in dimensions according to Table 4.4, where the final thickness of the steel plate was determined by the FLS-design for Björnbo Bridge.

Table 4.4: Dimensions Björnbo Bridge, Viacon profile VT0.5.

<table>
<thead>
<tr>
<th>Description</th>
<th>Pipe inner diameter</th>
<th>Determinant length of pipe</th>
<th>Pipe inner height</th>
<th>Height from springline to crown</th>
<th>Thickness steel plate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Notation</td>
<td>D</td>
<td>$L_p$</td>
<td>$H_{pipe}$</td>
<td>$H$</td>
<td>$t$</td>
</tr>
<tr>
<td>Unit</td>
<td>[mm]</td>
<td>[mm]</td>
<td>[mm]</td>
<td>[mm]</td>
<td>[mm]</td>
</tr>
<tr>
<td>Design value</td>
<td>2528</td>
<td>5056</td>
<td>2394</td>
<td>1260</td>
<td>6.0</td>
</tr>
</tbody>
</table>
Cross-sectional parameters for the steel pipe according to formulas provided in report 112 were determined according to Table 4.5.

Table 4.5: Cross-sectional parameters for Viacon profile VT0.5.

<table>
<thead>
<tr>
<th>Description</th>
<th>Area (mm²/mm)</th>
<th>Moment of inertia (mm⁴/mm)</th>
<th>Section modulus (mm³/mm)</th>
<th>Plastic bending resistance (mm³/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Notation</td>
<td>A</td>
<td>I</td>
<td>Wₘₐ</td>
<td>Wₘₙ</td>
</tr>
<tr>
<td>Unit</td>
<td>[mm²/mm]</td>
<td>[mm⁴/mm]</td>
<td>[mm³/mm]</td>
<td>[mm³/mm]</td>
</tr>
<tr>
<td>Design value</td>
<td>7.09</td>
<td>2659</td>
<td>87.2</td>
<td>126</td>
</tr>
</tbody>
</table>

Ultimate/Serviceability limit state verifications

The SLS verification involves analysing so high stresses are not developing in the wall of the pipe. This is performed by verifying so the steel in the analysis does not yield. The utilization rate for the stress in the wall of the pipe is estimated to 21%.

The utilization rates calculated for the flexural buckling of the crown are presented in Table 4.6. As can be seen, the utilization rates are below its full capacity and hence it can be concluded that the safety against flexural buckling is ensured.

Table 4.6: Utilization rates for flexural buckling of top crown (ULS).

<table>
<thead>
<tr>
<th>Control performed</th>
<th>Interaction normal force (Nₑₓₐ) and moment (Mₑₓₐ) equal to zero, Eurocode</th>
<th>Only normal force (Nₑₓₑ) and moment (Mₑₓₑ) equal to zero, Eurocode</th>
<th>Only normal force (Nₑₓₑ) and moment (Mₑₓₑ) equal to zero, BSK07</th>
</tr>
</thead>
<tbody>
<tr>
<td>Utilization rate</td>
<td>33%</td>
<td>16%</td>
<td>23%</td>
</tr>
</tbody>
</table>

For the bolted joints, the calculated utilization rates are presented in Table 4.7. The utilization rates are below its maximum capacity.

Table 4.7: Utilization rates for bolted joints (ULS).

<table>
<thead>
<tr>
<th>Control performed</th>
<th>Shear</th>
<th>Tension</th>
<th>Interaction of shear and tension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Utilization rate</td>
<td>24%</td>
<td>33%</td>
<td>40%</td>
</tr>
</tbody>
</table>

Fatigue assessment – Lambda method

The fatigue assessment is performed according to regulations in TRVK Bro and methods in report 112. The fatigue assessment is verified for the steel plate at certain locations, such as steel plate at the crown, steel plate at a bolted joint etc., of the SSCB. The fatigue assessment is based on the lambda-method, which is performed by determining factors to calculate a damage equivalent stress range. The factors are presented in Table 4.8. The $\lambda_{max}$ factor is the final correction factor for calculating damage equivalent stress range from the stress that is caused by the LM71-model multiplied by the DAF from SS-EN 1991-2, i.e. the factor $\phi$.

---

15 The input for stress category numbers are presented in Table 3.1.
16 Presented in Figure 3.2.
4.2. Static design

<table>
<thead>
<tr>
<th>Description</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Stress range factor that considers influence line length</td>
<td>Factor concerning traffic volume</td>
<td>Factor concerning the bridge lifetime</td>
<td>Factor concerning structural member that is subjected by several tracks on the bridge</td>
<td>Damage equivalent stress range factor</td>
<td></td>
</tr>
<tr>
<td>Notation</td>
<td>$\lambda_1$</td>
<td>$\lambda_2$</td>
<td>$\lambda_3$</td>
<td>$\lambda_4$</td>
<td>$\lambda_{\text{max}}$</td>
</tr>
<tr>
<td>Value</td>
<td>1.07</td>
<td>1.00</td>
<td>0.96</td>
<td>1.0</td>
<td>1.02</td>
</tr>
</tbody>
</table>

The presented calculated results are regarding fatigue for the different locations that should be considered for SSCB. The utilization rates for fatigue are presented in Table 4.9.

<table>
<thead>
<tr>
<th>Control performed</th>
<th>Steel plate at the crown (tension)</th>
<th>Steel plate near a bolted joint (tension)</th>
<th>Bolted joint (shear)</th>
<th>Bolted joint (tension)</th>
<th>Bolted joint (interaction between shear and tension)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Utilization rate</td>
<td>61%</td>
<td>99%</td>
<td>64%</td>
<td>61%</td>
<td>33%</td>
</tr>
</tbody>
</table>

It can be noted from Table 4.9 that the highest utilization rate is obtained at the material near a bolted joint. However, none of the utilization rates regarding FLS was above their maximum capacity. Thus, it can also be concluded that Björnbo Bridge will not suffer from fatigue failure during its lifetime.

The number of bolts that are needed in the design for Björnbo Bridge is designed according to FLS since it is the governing design. The results regarding number of bolts are presented in Table 4.10.

<table>
<thead>
<tr>
<th>Description</th>
<th>Number of rows with bolts</th>
<th>Centre distance</th>
<th>Bolts per meter length of pipe</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit</td>
<td>[n]</td>
<td>[mm]</td>
<td>[1/m]</td>
</tr>
<tr>
<td>Design</td>
<td>3</td>
<td>200</td>
<td>15</td>
</tr>
</tbody>
</table>

**Summary static structural design**

The structural design procedure of Björnbo Bridge is performed to analyse the structure for criterions regarding SLS, ULS and FLS. This is by analysing the structure with methods provided from report 112.

The SSCB fulfils criterions in SLS since the steel material of the pipe does not reach the state of yielding. For ULS criterions, the utilization rates are analysed for the crown of the pipe and bolted connections.

The FLS criterions is analysed at certain locations, Table 4.9, with the use of the Lambda method that is described in Eurocode documents. The highest utilization rate are obtained when analysing the stress near a bolted joint, 99 %. Which gave that the design for the bolted joints was governed by the FLS-design. Thus, the governing design state for Björnbo Bridge is the FLS, which also determined the final thickness of the steel plate to 6.0mm. As a comparison, if the final design state had been ULS, the
final thickness of the steel plate would have been set to 3.0mm. It should also be remembered that the designed thickness of the steel plate only assumes one railway track whilst the real case is that two railway tracks are located on the railway embankment of Björnbo Bridge.

Since the SSCB designed for Björnbo Bridge fulfil design criterions stated in design documents such as Eurocode (static load case), TRVR Bro and TRVK Bro and report 112. It can then be assumed that the SSCB is guaranteed to fulfil design requirements from a static structural design.

Thus, the design that has been guaranteed to meet static design requirements is in this thesis analysed in a dynamic analysis as well. The dynamic analysis is performed with a FE-simulation in an attempt to verify the SSCB against dynamic design method available in the Eurocode.

### 4.3 Dynamic analysis – Björnbo Bridge

**FE-model of the Björnbo Bridge**

The method of determining cross-section parameters by using a sub-model\(^{17}\) in Brigade/PLUS for the corrugation valid for Björnbo Bridge gave the following cross-section parameters as presented in Table 4.11.

<table>
<thead>
<tr>
<th>Notation</th>
<th>Young’s modulus (E (GPa))</th>
<th>Area (A (m(^2)))</th>
<th>Moment of inertia (I_{xx}) (m(^4))</th>
<th>Moment of inertia (I_{yy}) (m(^4))</th>
<th>Moment of inertia (I_{xy}) (m(^4))</th>
<th>Torsional constant (J (m(^4)))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brigade/PLUS</td>
<td>210</td>
<td>1.430·10(^{-3})</td>
<td>5.671·10(^{-7})</td>
<td>4.955·10(^{-6})</td>
<td>1.287·10(^{-6})</td>
<td>1.688·10(^{-8})</td>
</tr>
</tbody>
</table>

Table 4.11: Cross-section parameters Björnbo Bridge, corrugation 200x55.

In Figure 4.6, the geometric shape of the pipe and the full model with the discretized soil profile is presented and nodal result points for registration of accelerations.

The influence line for the crown displacement for Björnbo Bridge for a moving load of 100 kN is presented in Figure 4.7. Nodal results that are collected along the bridge profile in the FE-model regarding Björnbo Bridge is presented in Figure 4.8. The nodal results are collected from points presented in just mentioned figures. The same notations as from field measurement points from Märsta Bridge are used. For material properties and mesh properties, reference is given to Table 3.6 and Table 3.8 respectively.

---

\(^{17}\) The procedure of creating a sub-model of one corrugation is presented in chapter 3.3.
Damping/Mode shape analysis/Steady-state analysis

The composite modal damping estimated by Brigade/PLUS that is applied at each mode when performing the dynamic analysis for Björnbo Bridge is presented in Table 4.12.
Table 4.12: Modal composite damping for Björnbo Bridge.

<table>
<thead>
<tr>
<th>Mode (No.)</th>
<th>Frequency (Hz)</th>
<th>Composite modal damping (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9.95</td>
<td>2.26</td>
</tr>
<tr>
<td>2</td>
<td>10.29</td>
<td>2.36</td>
</tr>
<tr>
<td>3</td>
<td>11.52</td>
<td>2.30</td>
</tr>
<tr>
<td>4</td>
<td>14.30</td>
<td>2.36</td>
</tr>
<tr>
<td>5</td>
<td>16.12</td>
<td>2.43</td>
</tr>
<tr>
<td>6</td>
<td>16.34</td>
<td>2.30</td>
</tr>
<tr>
<td>7</td>
<td>19.17</td>
<td>2.36</td>
</tr>
<tr>
<td>8</td>
<td>21.23</td>
<td>2.34</td>
</tr>
<tr>
<td>9</td>
<td>22.24</td>
<td>2.52</td>
</tr>
<tr>
<td>10</td>
<td>23.31</td>
<td>2.52</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mode (No.)</th>
<th>Frequency (Hz)</th>
<th>Composite modal damping (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10.03</td>
<td>2.30</td>
</tr>
<tr>
<td>2</td>
<td>10.34</td>
<td>2.38</td>
</tr>
<tr>
<td>3</td>
<td>11.61</td>
<td>2.34</td>
</tr>
<tr>
<td>4</td>
<td>14.39</td>
<td>2.39</td>
</tr>
<tr>
<td>5</td>
<td>16.18</td>
<td>2.45</td>
</tr>
<tr>
<td>6</td>
<td>16.48</td>
<td>2.34</td>
</tr>
<tr>
<td>7</td>
<td>19.31</td>
<td>2.39</td>
</tr>
<tr>
<td>8</td>
<td>21.39</td>
<td>2.38</td>
</tr>
<tr>
<td>9</td>
<td>22.28</td>
<td>2.52</td>
</tr>
<tr>
<td>10</td>
<td>23.42</td>
<td>2.55</td>
</tr>
</tbody>
</table>

By performing an analysis of each mode shape excited by the frequencies, it is noted that there are small differences between analysing with high-density ballast or low-density ballast. Thus, only one set of mode shapes is presented in Figure 4.9, of which is connected with the high-density ballast. The analysis is performed for frequencies up to 30 Hz as following Eurocode recommendations. The mode shapes are presented with the surrounding soil to present the interaction between soil and SSCB-profile. Note that if there could be possibilities to estimate differently depending on the longitudinal size of the model for continuum elements.

Figure 4.9: Mode shape analysis for Björnbo Bridge.

The steady state analysis is performed with a point load of 200 kN applied at the railway track above the crown. The result showed that for Björnbo Bridge a total damping of 5.3% is achieved both when analysing Björnbo Bridge with high-density
and low-density of the ballast. The Half Power Bandwidth Method\textsuperscript{18} analysis for Björnbo Bridge with high-density ballast is presented in Figure 4.10.

![Steady state analysis](image1)

**Figure 4.10:** Steady-state response and Half Power Bandwidth Method for Björnbo Bridge.

**HSLM-analysis**

The largest dynamic response in terms of accelerations could be registered for the HSLM-A2 train-set. The maximum crown acceleration was estimated to 1.27 m/s\(^2\) from nodal point a1 at 290 km/h with high-density ballast. The maximum acceleration in the ballast was estimated to 1.26 m/s\(^2\) for the same configuration from nodal point a2 at 290 km/h. The acceleration analysis is presented in Figure 4.11.

![HSLM-analysis](image2)

**Figure 4.11:** Acceleration envelopes for Björnbo Bridge.

By analysing the contour plot from the max envelope from vertical accelerations, Figure 4.12, for Björnbo Bridge, with high-density ballast and HSLM-A2 train, it is

\textsuperscript{18} Presented in chapter 3.4, Figure 3.24 and Eq. (3.26).
noticed that the maximum acceleration is located in the region of the crown and reaches up to the ballast.

Figure 4.12: Contour plot of vertical acceleration from one passage of HSLM-A2, Märsta Bridge.

An estimation of possible velocities where the train and crown may be in resonance for Björnbo Bridge is presented in Table 4.13. To be noted is that resonance may occur when the travelling speed of the train is below the speed, 250 km/h, which is generally denoted as a high-speed railway.

Table 4.13: Possible velocities where the HSLM-A2 trainload and the crown of Björnbo Bridge may be in resonance.

<table>
<thead>
<tr>
<th>Resonance speed Figure 4.11 (km/h)</th>
<th>Resonance frequency Eq. (3.27) (Hz)</th>
<th>Frequency from steady-state analysis Figure 4.10 (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>135</td>
<td>10.7</td>
<td>10.0</td>
</tr>
<tr>
<td>180</td>
<td>14.3</td>
<td>14.3</td>
</tr>
<tr>
<td>290</td>
<td>23.0</td>
<td>21.4</td>
</tr>
</tbody>
</table>

The displacement envelope for all HSLM-A trains, with focus on the response from HSLM-A2, is presented in Figure 4.13. The largest displacement for the crown is estimated to 0.52 mm with high-density ballast at 290 km/h for HSLM-A2. The largest displacements in total are registered from HSLM-A9/10.

Figure 4.13: Displacement envelope for Björnbo Bridge.
The moment envelope with focus on HSLM-A2, the maximum moment are registered at 26.8 Nm and minimum moment at -12.8 Nm at 290 km/h for high-density ballast. All envelopes for the HSLM-trains are presented in Figure 4.14.

![Figure 4.14: Moment envelope for Björnbo Bridge.](image)

With the estimated moments and displacements for Björnbo Bridge and corresponding calculated static moments and displacements it is possible to evaluate DAFs for each HSLM-train. For the HSLM-A2 train the maximum DAF for displacement is estimated to 1.18 for high-density ballast. While the maximum DAF for the moment is estimated to 1.82. The estimated DAFs are presented in Figure 4.15.

![Figure 4.15: DAFs for Björnbo Bridge.](image)

Moreover, the special DAFs regarding FLS for Björnbo Bridge has been estimated with the fundamental frequency estimated for high-density ballast as given in Table 4.12. The estimated DAFs are presented in Table 4.14 with determinant length as suggested
in report 112 [29] and with the influence length of Björnbo Bridge from FEM-simulations.

Table 4.14: DAFs for FLS for Björnbo Bridge.

<table>
<thead>
<tr>
<th></th>
<th>$L_\phi$ (m)</th>
<th>$1 + 0.5 \varphi^n$ (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Report 112</td>
<td>5.17</td>
<td>1.131</td>
</tr>
<tr>
<td>Brigade/PLUS</td>
<td>16.5</td>
<td>1.152</td>
</tr>
</tbody>
</table>

The stresses that has been collected from FE-simulations is taken at the crest of the corrugation for the SSCB-profile. Comparisons have been made against the valley of the corrugation and the stress estimated at crest corrugation has consequently been estimated larger. The stress envelope is presented in Figure 4.16.

Figure 4.16: Stress envelope for Björnbo Bridge.

Fatigue assessment of the Björnbo Bridge

During this thesis, the fatigue analysis has been focused on the bolted joints that connects two steel plates together. The bolted joint is assumed to be located at the crown of the SSCB-profile and is analysed with $\Delta \sigma_c = 90$ MPa\(^{19}\). The dynamic stress-time history is amplified according to Eq. (3.18). The analysis of the stress signal is performed with the Palmgren-Miner\(^{20}\) rule by counting cycles for each stress range that is then combined to a stress collective. The stress collective from the stress signal is presented in Figure 4.17. Since the HSLSM-A2 train did not cause any cumulative damage from one passage, it can be assumed that Björnbo Bridge has infinite lifetime regarding FLS-analysis.

\(^{19}\) Note: The assumption of analysing the stresses at this location with $\Delta \sigma_c = 90$ MPa is not available in the current TRVK Bro.

\(^{20}\) Presented in chapter 3.2.2.
4.3. Dynamic analysis – Björnbo Bridge

Summary dynamic analysis

The dynamic analysis of the Björnbo Bridge according to methods provided in the Eurocode has shown that the Björnbo Bridge fulfils SLS criterions, estimated accelerations are below $3.5 \text{ m/s}^2$ for the ballasted track. Björnbo Bridge analysed in FLS with the Palmgren-Miner rule has shown that for one passage of the HSLM-A2 train, no cumulative damage was obtained by analysing the stress collective. Thus, the Björnbo Bridge will not suffer from fatigue.

By referring to Table 3.9, it has been observed that the FE-model has not been able to estimate correct accelerations when comparing to field measurement for the X52-train. This indicates that the accelerations could in fact be larger when analysing HSLM-trains if the created FE-model has been more accurate in estimating accelerations. This indicates that the created FE-model suffers from un-certainties in its performance regarding estimating accelerations during HSLM-simulations. Regarding dynamic displacements and stresses, it has been possible to confirm that the FEM-model can estimate displacements and strains within reasonable margins.

Moreover, there are un-certainties regarding how much damping that is achieved during simulations with the use of composite modal damping since the analysis with the use of the Half Power Bandwidth Method has shown that that the damping is larger than values obtained from Brigade/PLUS, see Table 4.12 and Figure 4.10.
Chapter 5

Parametric study on the dynamic performance of SSCB

A parametric study of SSCB in general has been performed both with report 112 for static structural design principles and FEM-simulations for dynamic principles. The parametric cases that are investigated are described first for static structural analysis, chapter 5.1, and then for dynamic analysis, chapter 5.2. The focus are of presented results from dynamic parametric studies from FEM-simulations. The train that is used is the HSLM-A2 train. The parametric results from static structural analysis described in report 112 are presented in Appendix B. The results from the dynamic parametric studies are presented in chapter 5.3 with comments.

5.1 Static design

The parametric studies conducted according to the methods prescribed in report 112 are performed according to pre-determined schemes and is mainly performed to ensure that the static design is verified. Moreover, some analyses of surrounding stresses adjacent to the pipe is investigated to see if large variations occur for different configurations of pipe-dimensions and soil cover depth. Since the method is already developed in report 112, a computer-aided calculation-sheet from Mathcad software is at hand. The following describes the parametric analysis that will be performed using the Mathcad-sheet.

The governing parameters that are free to adjust can be concluded to:

- Soil cover depth, $h_c$
- Profile type, VT or VE including changes of span length

Parameters that are affected by the change of the above mentioned parameters and affect the design values obtained for normal forces and moments in report 112 is concluded to:
Dynamic Amplification Factor (\( \phi \)) determined from the Eurocode by using determining lengths that is determined from the choice of profile.

Dynamic reduction factor suggested in report 112 is governed by \( h_c \). Hence, this factor will be determined according to the choice of soil cover depth.

Combined stiffness ratio, \( \lambda_f \), is affected as new values for the tangent modulus \( (E_{sd}) \) and flexural stiffness for the pipe \( ((EI)_f) \) is obtained.

### Soil cover depth

When analysing different soil cover depth, it will be in the range of 1.1m-3.0m with a variation in five steps with \( \Delta h_c = 0.38m \). This will have the effect of giving different stress levels at the analysed depth in the soil and affecting the calculated tangent modulus \( (E_{sd}) \) as obtained in Eq. (3.1) from report 112. The other parameters are constant and have the following configuration:

- Degree of compaction, \( RP = 97\% \). An assumption is made that a reasonable high value is obtained.
- Profile type, the profile type is set to the VT0.5 profile.

### Profile type and span length

The change of profile type provides many geometric changes for the pipe used in the design. Thus, the effect that will be seen is if the different shape of pipes affects the final static response. The change of profile type will give different total height calculated from the spring line of the pipe to the bottom edge of the railway sleepers. In addition, a change of the span length, i.e. the SSCB diameter \( D \), is combined with change of profile type. The changes of profile type can be presented as:

- VT-profile: Studies of span length in ranges of 2m-6m with variation of span length as \( \Delta D \geq 0.5m \). This gives that the following profiles are evaluated; VT0.5, VT4, VT7, VT13 and VT17.
- VE-profile: Studies of span length in ranges of 2m-3.5m with variation of span length as \( \Delta D \geq 0.3m \). This gives that the following profiles are evaluated; VE4, VE6, VE8, VE10 and VE12.

The input and name for each profile is presented in Table 5.1-Table 5.3 below. The same profile input is used for the parametric dynamic analysis. For a description of the notation, reference is given to Figure 3.1.

### Table 5.1: VT-profiles valid for report 112, profile geometry.

<table>
<thead>
<tr>
<th>Notation</th>
<th>( D_i ) [m]</th>
<th>( H_i ) [m]</th>
<th>Height from spring line [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>VT0.5</td>
<td>2.528</td>
<td>2.394</td>
<td>1.260</td>
</tr>
<tr>
<td>VT4</td>
<td>3.395</td>
<td>3.105</td>
<td>1.700</td>
</tr>
<tr>
<td>VT7</td>
<td>3.855</td>
<td>3.485</td>
<td>1.925</td>
</tr>
<tr>
<td>VT13</td>
<td>5.035</td>
<td>4.745</td>
<td>2.520</td>
</tr>
<tr>
<td>VT17</td>
<td>5.915</td>
<td>5.425</td>
<td>2.960</td>
</tr>
</tbody>
</table>
5.1. STATIC DESIGN

Table 5.2: VT-profiles valid for report 112, profile radius and angles.

<table>
<thead>
<tr>
<th>Notation</th>
<th>Top radius [m]</th>
<th>Corner radius [m]</th>
<th>Bottom radius [m]</th>
<th>Top angle [˚]</th>
<th>Corner angle [˚]</th>
<th>Bottom angle [˚]</th>
</tr>
</thead>
<tbody>
<tr>
<td>VT0.5</td>
<td>1.289</td>
<td>0.967</td>
<td>2.846</td>
<td>242.7</td>
<td>50.2</td>
<td>16.8</td>
</tr>
<tr>
<td>VT4</td>
<td>1.697</td>
<td>1.050</td>
<td>3.637</td>
<td>234.28</td>
<td>50</td>
<td>25.73</td>
</tr>
<tr>
<td>VT7</td>
<td>1.929</td>
<td>1.050</td>
<td>5.625</td>
<td>240.94</td>
<td>50</td>
<td>19.06</td>
</tr>
<tr>
<td>VT13</td>
<td>2.518</td>
<td>1.858</td>
<td>4.895</td>
<td>238.12</td>
<td>50</td>
<td>21.88</td>
</tr>
<tr>
<td>VT17</td>
<td>2.960</td>
<td>1.858</td>
<td>6.280</td>
<td>234.38</td>
<td>50</td>
<td>25.62</td>
</tr>
</tbody>
</table>

Table 5.3: VE-profiles valid for report 112, Profile geometry, radius and angles.

<table>
<thead>
<tr>
<th>Notation</th>
<th>Inner diameter [m]</th>
<th>Inner height [m]</th>
<th>Height from springline [m]</th>
<th>Top radius [m]</th>
<th>Side radius [m]</th>
<th>Top angle [˚]</th>
<th>Side angle [˚]</th>
</tr>
</thead>
<tbody>
<tr>
<td>VE4</td>
<td>2.075</td>
<td>2.305</td>
<td>1.153</td>
<td>0.963</td>
<td>1.243</td>
<td>95.2</td>
<td>84.8</td>
</tr>
<tr>
<td>VE6</td>
<td>2.505</td>
<td>2.775</td>
<td>1.388</td>
<td>1.143</td>
<td>1.463</td>
<td>80.6</td>
<td>99.4</td>
</tr>
<tr>
<td>VE8</td>
<td>2.795</td>
<td>3.085</td>
<td>1.543</td>
<td>1.273</td>
<td>1.633</td>
<td>82.8</td>
<td>97.2</td>
</tr>
<tr>
<td>VE10</td>
<td>3.225</td>
<td>3.715</td>
<td>1.858</td>
<td>1.443</td>
<td>1.863</td>
<td>73.0</td>
<td>107</td>
</tr>
<tr>
<td>VE12</td>
<td>3.525</td>
<td>3.875</td>
<td>1.938</td>
<td>1.523</td>
<td>2.003</td>
<td>60.6</td>
<td>119.4</td>
</tr>
</tbody>
</table>

Thus, providing changes as well in many parameters, the changes will be in the stress levels which affects the tangent modulus ($E_{sd}$) in Eq. (3.1). The combined stiffness ratio of the pipe and soil in Eq. (3.2) will be affected. During the change of profile types, the following parameters have the following configuration:

- Soil cover depth, $h_c = 1.1$m. The minimum value that is allowed as stated in TRVR Bro.
- Degree of compaction, $RP = 97\%$. An assumption is made that a reasonable high value is obtained.
- Steel thickness of the plate is set to $t = 6.0$ mm.

**Result presented from parametric studies – Report 112**

The compiled result from the above-mentioned configurations of parametric studies will be presented in suitable graphs presenting variation in stresses adjacent to the different configurations of pipe-dimensions and variation in utilization rates calculated from the verification methods presented in report 112. This is to ensure that the configurations that will be analysed in the finite element environment for dynamic load cases also fulfill criterions stated in report 112 regarding static load cases.

Since the aim in this report is to focus on the dynamic analysis, the compiled results from the parametric studies performed with methods from report 112 is presented in Appendix B with brief comments.
5.2 Dynamic design

Soil cover depth, $h_c$

The soil cover depth in the dynamic response analysis will follow the same range as determined for the parametric studies in report 112, i.e. 1.1-3.0 m. This will make it apparent to see the differences that can be expected between static and dynamic analysis for the same range of changes in soil cover depth. Depending on the differences in results from analysis of minimum and maximum values of soil cover depth, an interval is later decided during the analysis.

Young’s modulus of the engineered backfill is set to 220 MPa, as it was determined during the calibration process of the reference model. The profile during this analysis is the VT0.5 profile with corrugation 200x55.

Young’s modulus of the soil, $E_{sd}$

Young’s modulus of the soil is affecting the combined stiffness of the soil and the SSCB, it is important that it be determined correctly to obtain the correct solution. In this thesis, a dynamic modulus will be modelled since it has been proven earlier in other cases, that when a train passes the soil increases in stiffness. To model a dynamic modulus means that it is assumed that the structure under analysis is only subjected by short-term loading, which means a higher modulus can be utilized when analysing the structure. The dynamic modulus will in this case be modelled in ranges between 50-300 MPa, where first the minimum and maximum modulus will be analysed. If it is shown that the stiffness have large effect on the dynamic response, the range will be divided in intervals to analyse the differentiation in dynamic response.

The soil cover depth is set to 1.57m and the profile used during this analysis is the VT0.5 profile with corrugation 200x55.

Profile types, VT & VE

It is of interest to see what the differences in different profiles can be expected in terms of accelerations and frequencies for the dynamic analysis since the different profiles has different values of flexural stiffness. To start with, the same profiles that was analysed in report 112 for the minimum size and maximum size, i.e. VT0.5, VT17, VE4 and VE12, of each profile-type will be analysed in the FE-environment. If it is seen that large differences in response is obtained, further analysis will be made by analysing the remaining profile sizes as described in the parametric studies for report 112. In Figure 5.1, the maximum and minimum VT- and VE-profile is presented in a simplified figure that describes the differences between pipe-sizes in FE-models. For more detailed information about SSCB-dimensions, which is used in the parametric studies, reference is given to Table 5.1-Table 5.3.

The corrugation is set to 200x55. The surrounding environment, i.e. stiffness of engineered backfill and soil cover depth, is set as 220 MPa and soil cover depth as 1.57m. Other parameters, such as material definitions and mesh properties, are as defined in Table 3.6 and Table 3.8.
5.3 Parametric study of the dynamic response

Note that when the parametric analysis has been performed for the profile called ‘VE-Marsta’ it has been performed for a soil cover depth of 1.57m. The ‘VE-Marsta’ profile is the profile from Märsta Bridge.

Acceleration crown and ballast

When analysing the acceleration response at the crown for the different cases, the largest response is occurring for the case with lowest Young’s modulus for the soil with acceleration estimated to 2.3 m/s² at 300 km/h, Figure 5.2d). The different cases for the acceleration at the crown are presented in Figure 5.2.

Figure 5.2: Acceleration response at crown.
The acceleration at the level of the ballast is presented in Figure 5.3. The maximum value is registered for the case with lowest Young’s modulus for the soil with acceleration in the ballast-layer estimated to 2.3 m/s² at 300 km/h, Figure 5.3d).

A summarized table of all the maximum accelerations observed is presented in Table 5.4.

The effect of the decreasing stiffness resulting in increasing accelerations, Figure 5.2d) & Figure 5.3d), was not an expected result. This indicates that the choice of Young’s modulus as a dynamic modulus needs more investigation how to be estimated correctly. In this thesis, the final Young’s modulus of the engineered backfill has been chosen only with respect to calibrate deformations. For the time this research is performed, there are no current methods for how to estimate a dynamic modulus without the help of earlier field measurements.

The magnitude of the estimated accelerations does not vary when studied in the level of the ballast or at the level of the crown. This indicates that the shape of the profile has very little influence on the magnitude of vertical accelerations when studying SSCB for the HSLM-trains. However, analysing the vertical acceleration envelope signal from the figures shows that for smaller profiles, the acceleration peak has moved to higher velocities, Figure 5.2a) and Figure 5.3a). This is believed to be the effect of smaller profiles providing in total a stiffer structure.

The increasement of soil cover depth has the effect of lowering the accelerations. The cause for this behaviour is believed to be the spreading of the vertical acceleration by the depth of the discretized soil-profile when studying accelerations at the level of the crown.

The parameter that has yielded largest sensitivity when estimating acceleration can be concluded to the Young’s modulus of the engineered backfill.
Table 5.4: Observed maximum values for accelerations.

<table>
<thead>
<tr>
<th>VT-profiles</th>
<th>VE-profiles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>VT-profiles</td>
</tr>
<tr>
<td>VT0.5</td>
<td>1.3</td>
</tr>
<tr>
<td>VT7</td>
<td>1.4</td>
</tr>
<tr>
<td>VT17</td>
<td>1.4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Crown</th>
<th>Ballast</th>
</tr>
</thead>
<tbody>
<tr>
<td>VT0.5</td>
<td>1.3</td>
</tr>
<tr>
<td>VT7</td>
<td>1.4</td>
</tr>
<tr>
<td>VT17</td>
<td>1.4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>VT-profiles</th>
<th>VE-profiles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>VT-profiles</td>
</tr>
<tr>
<td>VE4</td>
<td>1.1</td>
</tr>
<tr>
<td>VE12</td>
<td>1.3</td>
</tr>
<tr>
<td>VE-Marsta</td>
<td>1.4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Crown</th>
<th>Ballast</th>
</tr>
</thead>
<tbody>
<tr>
<td>VT-profiles</td>
<td>VE-profiles</td>
</tr>
<tr>
<td>VE4</td>
<td>1.1</td>
</tr>
<tr>
<td>VE12</td>
<td>1.3</td>
</tr>
<tr>
<td>VE-Marsta</td>
<td>1.4</td>
</tr>
</tbody>
</table>

By analysing the signals of both acceleration at the crown and in the ballast, Figure 5.2c) and Figure 5.3c), they are in principal identical with each other. This indicates that the acceleration limits are the same in the in the ballast and in the level of the crown when analysis is performed with HSLM-trains, Table 5.4. However, whether this behaviour is realistic or not is not studied in detail in this thesis. The reason for this behaviour is believed to be the performance from the HSLM-trains, which is giving largest acceleration in midspan of the railway beam. What possibly could be expected is that the acceleration is estimated larger in the ballast layer and lower in the crown of the SSCB, because of spreading of the vertical acceleration in the soil before reaching the crown.

**DAF from deformations and moments**

The DAF that is determined as recommended from the Eurocode is presented in Figure 5.4. The largest DAF is determined for the case with the lowest Young’s modulus for the soil and is estimated to 1.56 at 300 km/h, Figure 5.4d), which indicates that the load should be increased by 56%.
Figure 5.4: DAF from displacements at the crown.

The DAF that is determined from moments as a scientific comparison against the DAF recommended from Eurocode is presented in Figure 5.5. The largest DAF is determined to 3.26 at 300 km/h, Figure 5.5d), for the case with lowest stiffness of the Young’s modulus for the soil. This means in practice that the load would be increased by 226 % if the Björnbo Bridge were designed according to DAFs determined from moments.

Figure 5.5: DAF from moments at the crown.

The observed maximum DAFs for both displacement and moments are summarized in Table 5.5.

The calculated DAFs for the different profiles are not following any specific trend when analysing the magnitude of the DAFs. By analysing the signal of the DAFs,
5.3. Parametric study of the dynamic response

Figure 5.4a) & Figure 5.4b) and Figure 5.5a) & Figure 5.5b), the conclusion is that resonance peaks for moments and displacements may be shifted from each other.

Analysing the DAFs for different Young’s modulus indicates that for a lower stiffness of the engineered backfill, the larger the DAF is estimated. This behaviour is believed to be the cause of setting a high modulus, it gives that the displacements and moments are estimated smaller. Thus, the fraction between dynamic response and static response is larger which results in a lower magnification factor to consider for the dynamic behaviour. However, the DAF regarding moments is estimated to be very large, even though the moments has been estimated small. It can also be seen that when Young’s modulus is set equal to 50 MPa, valid in static design methods, the acceleration shows an increasing trend for speeds above 200 km/h.

When analysing the magnitude of the DAFs for displacements, it can be concluded that it follows a decreasing trend. This is believed to be because of load spreading by the depth of the discretized soil-profile. The trend for DAF estimated from moments is decreasing for increased soil cover depth.

This behaviour of differences between DAFs for displacements and moments may cause difficulties when performing designs for SSCB for the practising engineer to create a design that suits every aspect. However, in the current Eurocode documents, it is assumed sufficient to only determine a DAF for displacements.

Table 5.5: Observed maximum values for DAF.

<table>
<thead>
<tr>
<th>VT-profiles</th>
<th>VE-profiles</th>
</tr>
</thead>
<tbody>
<tr>
<td>VT0.5</td>
<td>VE4</td>
</tr>
<tr>
<td>VT7</td>
<td>VE12</td>
</tr>
<tr>
<td>VT17</td>
<td>VE-Marsta</td>
</tr>
<tr>
<td>Displacement</td>
<td>1.18</td>
</tr>
<tr>
<td>Moment</td>
<td>1.82</td>
</tr>
<tr>
<td>Displacement</td>
<td>1.12</td>
</tr>
<tr>
<td>Moment</td>
<td>1.67</td>
</tr>
<tr>
<td>Displacement</td>
<td>1.12</td>
</tr>
<tr>
<td>Moment</td>
<td>1.81</td>
</tr>
<tr>
<td>Soil cover depth</td>
<td>1.1 (m)</td>
</tr>
<tr>
<td>Displacement</td>
<td>1.23</td>
</tr>
<tr>
<td>Moment</td>
<td>2.26</td>
</tr>
<tr>
<td>Displacement</td>
<td>1.18</td>
</tr>
<tr>
<td>Moment</td>
<td>1.82</td>
</tr>
<tr>
<td>Displacement</td>
<td>1.14</td>
</tr>
<tr>
<td>Moment</td>
<td>1.82</td>
</tr>
<tr>
<td>Displacement</td>
<td>1.14</td>
</tr>
<tr>
<td>Moment</td>
<td>1.63</td>
</tr>
<tr>
<td>Young’s modulus</td>
<td>1.57 (m)</td>
</tr>
<tr>
<td>Displacement</td>
<td>1.18</td>
</tr>
<tr>
<td>Moment</td>
<td>1.82</td>
</tr>
<tr>
<td>Displacement</td>
<td>1.11</td>
</tr>
<tr>
<td>Moment</td>
<td>1.63</td>
</tr>
</tbody>
</table>

Frequencies

The frequencies that are estimated for each parameter are presented in Figure 5.6. Since the current design methods does not include SSCB for pre-determining frequencies and connected mode shapes. A collection of the different frequencies excited for each parameter are presented in Figure 5.6 for the purpose of determine which parameter that affect the frequencies the most. As can be told, the largest influence on the excited frequency is when analysing Young’s modulus for the engineered backfill.
CHAPTER 5. CASE STUDY OF THE BJÖRNBO BRIDGE

Figure 5.6: Estimated frequencies during parametric studies.

Regarding frequencies for different Young’s modulus, since the stiffness of the total system is increasing the estimated frequencies for the system is affected. Since the stiffness matrix of the model has changed, the eigenvalues is calculated differently when solving the eigen problem for the \( n \)th natural frequency in FEM-simulations.

Thus, if different Young’s modulus would be accepted when using static design methods in report 112 and dynamic design methods in FEM-software it would mean that different estimated frequencies could be expected in the separate design procedures for the same structural system.
Chapter 6

Discussion and conclusion

6.1 Model insecurities

The results regarding accelerations from the equivalent 2D-models have shown that it has not been possible to estimate the correct acceleration compared to field measurements, Table 3.9. Thus, the results that has been presented for acceleration for Björnbo Bridge and Märsta Bridge in the dynamic analysis for each bridge is not reliable. The reason for this is believed to be not being able to model the correct mass-properties, damping properties and Young’s modulus for engineered backfill.

The presented results for strains are both smaller and larger than the observed field measurements, Table 3.10. The reason for this is believed to be the assumption of effective width for one corrugation of the steel plate in FE-analysis that is not sufficient to be able to estimate the correct strains. These insecurities may have affected the FLS-analysis performed with the Palmgren-Miner rule. However, the static structural design has shown that the governing design state is FLS. Thus, the Björnbo Bridge is guaranteed to fulfil criterions for FLS-analysis according to the Lambda method.

6.2 Discrepancies: Märsta Bridge and Björnbo Bridge

Since the field measurements for Märsta Bridge is compared to estimated values for the Björnbo Bridge, it should be commented that there are differences in-between the two SSCB regarding several factors.

The Märsta Bridge is assumed to be founded directly on solid rock whilst Björnbo Bridge is assumed to be founded on moraine with relatively high strength according to technical documents valid for Björnbo Bridge. This means in practice that the geological conditions are different for the two bridges. Mainly the soil depth to solid rock. Thus, this could have an effect on calculated deformations, accelerations, chosen boundary conditions, frequencies etc.
Märsta Bridge is of the kind VE-profile with corrugation 150x50 and Björnbo Bridge is going to be constructed with a VT-profile with corrugation 200x55. Since there are differences in the geometrical appearances in-between the profiles and corrugation types, differences can be found in cross-section parameters. This would mean that the total stiffness of the SSCB is different and again affecting the deformations.

There can be differences for the mass-properties of the soil material adjacent to the SSCB; this would have an effect on the accelerations. Moreover, the soil cover depth differs between the two bridges. For Björnbo Bridge, the estimated soil cover depth from drawings is 1.57m from the crest of the corrugation to bottom of the railway sleepers. For Märsta Bridge, the soil cover depth was measured to 1.7m when collecting the field measurements.

### 6.3 Model calibration

The reference model was calibrated against field measurements collected for Märsta Bridge to be able to guarantee that the 2D FE-model estimated the correct results. The process of calibrating the 2D FE-model against field measurements is good for research purposes. However, in a situation where an SSCB would be designed in a dynamic analysis, it would be impossible to guarantee if the FE-model actually fulfils dynamic SLS-criterions, \( a_{\text{max}} \leq 3.5 \text{ m/s}^2 \), since there are insecurities in how to model surrounding environment that has shown in this thesis is crucial when estimating acceleration levels. Thus, more focus should be on how to create individual models that generate the same results for acceleration and other entities.

### 6.4 Dynamic analysis – Björnbo Bridge

#### Frequencies & Mode shapes

Since there are yet not methods available to predict the frequencies and connected mode shapes for SSCB other than by FE-simulations, it is not possible to state whether these frequencies should be regarded as existing\(^{21}\) frequencies. This indicates that there are still unknown phenomenon regarding the dynamic behaviour of SSCB and how these should be modelled and be represented correctly in FE-simulations.

The mode shapes for Björnbo Bridge, presented in Figure 4.9, shows that the surrounding soil is highly interactive with the SSCB. This makes it difficult to tell which of the mode shapes that is connected solely with the SSCB. To analyse which of the frequencies that has excited a mode shape solely connected with the SSCB, the use of steady-state analysis has been deemed a possible approach, presented in Figure 4.10 for Björnbo Bridge.

---

\(^{21}\) This means as if the frequency actually is existing for a completed SSCB when it is in use as a railway-bridge.
6.5 Parametric studies on dynamic load cases for SSCB-structures

6.5.1 Influence from parameters in dynamic analysis

The behaviour of different parameters effect on dynamic response has been a part to analyse in this thesis. The parameter that has been analysed is; profile type (VE- and VT-profiles), soil cover depth and Young’s modulus for the engineered backfill. The studies have been performed for the HSLM-A2 train. The studied entities have been confined to vertical acceleration at the crown, vertical acceleration at the level of the ballast, DAFs from displacement and DAFs from moments.

Summary – Conclusions

From chapter 5.3, the following conclusions is drawn;

- For larger soil cover depth, the acceleration is estimated lower in the crown. However, this behaviour has also been registered for acceleration limit in the ballast. Thus, it is questionable how well the accelerations are estimated in the parametric studies in this thesis with a 2D FE-model.

- The estimated DAFs for both displacements and bending moment decreases with larger soil cover depth. However, the DAF can still be estimated large for moments for different soil cover depths when compared to DAF estimated for displacements, see Table 5.5.

- The choice of Young’s modulus highly affects the estimated accelerations. For a lower Young’s modulus, the acceleration increases. This behaviour is believed to be the effect of a decreasing Young’s modulus affect the magnitude of accelerations in vertical direction in continuum bodies for the railway embankment studied in the case for SSCB in 2D FE-models. Moreover, the spreading of the vertical acceleration by the depth of the discretized soil volume may be effected by increasing the stiffness of the soil. This effect has not been studied in detail in this thesis.

- The DAFs for displacements and bending moments are affected by the choice of the Young’s modulus. When increasing the Young’s modulus, DAFs decreases.
The estimated frequencies and mode shapes is highly affected by the choice of Young’s modulus. More research is needed regarding the choice of Young’s modulus for the engineered backfill to be able to establish a consistent Young’s modulus that can be used in both static design principles and dynamic design analysis.

For each parameter that has been investigated in this thesis, it has been noticed that when velocities above 200 km/h is analysed, the dynamic behaviour for SSCB is clearly increasing. Thus, it can be concluded that if SSCB would be designed for high-speed railways, the design should mainly be focused on the higher design speeds.

6.5.2 Dynamic Amplification Factors

Referring to Table 5.5 it is seen that there is a clear difference between the two determined DAFs, with the clearest difference in the magnitude. Other differences that can be observed are the location of peaks, Figure 4.15, which indicates that there can be differences when resonance is occurring for moments and resonance occurring for displacements.

However, the most crucial influence is the magnitude since the maximum value is going to fulfil a magnification factor of the loads used in design. Thus, if the factor is too large it may lead to over-estimation of loads, which further lead to increasing the usage of material by increasing the steel thickness for SSCB to fulfil safety requirements and other verifications for structural members. This may lead to un-economic structures and thus losing one favourable aspect to design a SSCB compared to a portal-frame bridge that may be more suitable from a dynamic perspective.

Moreover, when determining a DAF for moments, Eq. (3.14), it is important to investigate both negative and positive moments and determine a DAF for both since the magnitude of the positive moments and negative moments can be different. The final DAF should be taken as the larger of the two. In this thesis, the DAF from negative moments has generated largest DAFs.

If the suggestion would be given to a practising engineer to determine a DAF from moments. There should be specific limitations that the factor should not be allowed to estimate too large. Since there can be deviations in FE-models depending the designer who has constructed the model and other factors.

Regarding the DAF determined for displacements, presented in Figure 4.15, they can be considered to be estimated within a satisfying manner. This means that it would give an increase in the load that is assumed reasonable in a real design project and not leading to have too much increase of loads that leads to non-economic structures and un-favourable design situations for the practising engineer designing a SSCB.

Some conclusions are;
Determining a DAF for moments is that there are un-certainties if the DAF gives a too large magnification factor compared to the one determined for displacements.

In a practical point of view, the DAF for displacements is easier in the sense that since it is often easier to determine in advance, where it should be studied. Compared to bending moments in a composite structure like the SSCB. Thus, this would lead to not face the situation of difficulties while designing civil engineering structures.

### 6.6 Future research

Some suggestions for further research that might be of interest could be:

- **Further parametric studies;**

  There should be more parametric studies with in-depth analysis of the choice of Young’s modulus since it has proven this parameter affect many factors regarding SSCB.

  Damping procedures, there should be more analysis performed evaluating different kind of damping procedures to state which damping, and how much damping, that can be allowed when analysing SSCB from a dynamic perspective.

  Include box-culverts in a parametric study; perform FE-analysis of all types of SSCB-profiles that are included in report 112.

- **3D FEM-analysis with support from field measurements;**

  More analysis should be performed with 3D FE-analysis to increase understanding for SSCB with support from field measurements. This would make it easier to continue the research on how to simplify FE-simulations to equivalent 2D-models where dynamic analysis is performed. In addition, 3D-models might be needed since equivalent 2D-models cannot visualize structural behaviour such as twisting mode shapes.

- **More detailed fatigue analysis;**

  More detailed fatigue analysis should be performed for SSCB where more detailed modelling of the corrugation is performed since it has a large effect on the resulting stiffness to the structure. Moreover, it should involve more studied locations where fatigue might be needed to consider.

- **Dynamic analysis;**

  The presented research in this thesis could be performed again for a different set of profile types that Viacon present in their standard library of SSCB. A suggestion is however that the study should involve an existing bridge with existing field measurements, if possible, to delimit the static design procedure.
Moreover, the study should involve both 3D- and 2D-models to be able to determine the effective width for plain-strain thickness used in the equivalent 2D-model. This dynamic analysis could involve the suggestion of parametric studies for; Young’s modulus and damping procedures.
Bibliography


Appendix A

Static design of Björnbo Bridge

Static design calculation and verifications for Björnbo Bridge according to design principles from report 112 developed at Sweco in Mathcad.

Some smaller hand-calculations have been performed according to the methods from report 112 to makes sure that the Mathcad-sheet provides correct results. The hand calculations are also provided. The design calculations are performed by Joakim Woll.

In the calculation, the Björnbo Bridge is called ‘Skutskär Bridge’, which should be recognized as the Björnbo Bridge located in Skutskär.
Verification of MathCAD sheet

A calculation of the cross-section parameters, corrugation details and the Tangent modulus according to method B is performed to verify that the mathcad-routine developed at Sweco generates the same results.

SSCB: Björnbo Bridge
Profile: VT0.5
Corrugation: 200x55 [mmxmm]

**Input**

- Top radii: \( R_t := 1.289\text{-m} \)
- Corner radii: \( R_c := 0.967\text{-m} \)
- Bottom radii: \( R_b := 2.846\text{-m} \)
- Height springline: \( H_{spr} := 1.260\text{-m} \)
- Total height: \( H_{tot} := 2.394\text{-m} \)
- Diameter: \( D := 2.528\text{-m} \)
- Thickness plate: \( t := 6.0\text{-mm} \)
- Height corrugation: \( h := 55\text{-mm} \)
- Wavelength: \( c := 200\text{-mm} \)

**Corrugation data**

\[
\alpha_{\text{rad}} := 0.759 + 0.010 \frac{t}{\text{mm}} = 0.82\text{-rad}
\]

\[
\alpha := \frac{\alpha_{\text{rad}} \cdot 360\text{-deg}}{2\cdot\pi} = 47\text{-deg}
\]

\[
mt := 37.5\text{-mm} - 1.83\cdot t = 27\text{-mm}
\]

\[
R_{53} := 53\text{-mm}
\]

\[
r := R_{53} + \frac{t}{2} = 56\text{-mm}
\]

\[
e_c := r \left( 1 - \frac{\sin(\alpha_{\text{rad}})}{\alpha_{\text{rad}}} \right) = 6.1\text{-mm}
\]
Cross-section parameters

Moment of inertia

\[ I := \left( \frac{3}{4} \cdot \alpha_{\text{rad}} \cdot \left( \frac{h}{2} - e_c \right) + \frac{1}{2} \cdot \left( \frac{t}{\sin(\alpha_{\text{rad}})} \right) \left( m \cdot \sin(\alpha_{\text{rad}}) \right)^2 \right) \cdot \frac{4}{12} \cdot \alpha_{\text{rad}} \cdot r \cdot t \cdot h_2 \cdot e_c \quad \begin{bmatrix} \frac{1}{c} \end{bmatrix} = 2659 \text{ mm}^4 \text{ mm} \]

Plastic section modulus

\[ Z := \left( \frac{4 \cdot \alpha_{\text{rad}} \cdot r \cdot t \left( \frac{h}{2} - e_c \right)}{c} + \frac{1}{2} \cdot \left( \frac{t}{\sin(\alpha_{\text{rad}})} \right) \left( m \cdot \sin(\alpha_{\text{rad}}) \right)^2 \right) \quad \begin{bmatrix} \frac{1}{c} \end{bmatrix} = 126 \text{ mm}^3 \text{ mm} \]

Section modulus

\[ W_{\text{el}} := \frac{2 \cdot I}{h + t} = 87 \text{ mm}^3 \text{ mm} \]

Area

\[ A := \left( \frac{4 \cdot \alpha_{\text{rad}} \cdot r \cdot t + 2 \cdot m \cdot t}{c} \right) = 7.09 \text{ mm}^2 \text{ mm} \]

Tangent modulus - Method B

With values in Table B2.1 in report 112. The following tangent modulus is obtained for Björnbo Bridge. Calculated with \( d_{50} := 20 \text{ mm} \) and \( c_u := 10 \).

Optimum density (base course) Compact density

\[ \rho_{\text{opt}} := 20.6 \text{ kN/m}^3 \quad \rho_s := 26 \text{ kN/m}^3 \]

Standard Proctor scale Material density

\[ \text{RP} := 97\% \quad \rho := \frac{\text{RP}}{100\%} \cdot \rho_{\text{opt}} = 19.98 \text{ kN/m}^3 \]

Void ratio Stress exponent

\[ e := \frac{\rho_s}{\rho} - 1 = 0.3 \quad \beta := 0.29 \cdot \log \left( \frac{d_{50}}{0.01 \text{ mm}} \right) - 0.065 \cdot \log (c_u) = 1 \]

Modulus ratio

\[ m := 282 \cdot c_u \cdot e^{-2.83} = 1430 \]
Characteristic friction angle

$$\varphi_k := 26\, \text{deg} + 10 \left( \frac{\text{RP} - 75\%}{25\%} \right) \text{deg} + \left( 0.4 \cdot c_u \right) \text{deg} + \left( 1.6 \cdot \log \frac{d_{50}}{\text{mm}} \right) \text{deg} = 41\, \text{deg}$$

Vertical stress in soil

$$\sigma_1 := 37\, \text{kPa}$$

$$\sigma_1 = \rho \left( h_{c,\text{min}} + \frac{H}{2} \right)$$

Horizontal stress

$$\sigma_3 := \sigma_1 \left( 1 - \sin(\varphi_k) \right) = 13\, \text{kPa}$$

Tangent modulus, characteristic

$$\gamma_k := 1.0$$

$$R_f := 0.7$$

$$P_a := 100\, \text{kPa}$$

$$k_v := \frac{\sin(\varphi_k) \left( 3 - 2 \cdot \sin(\varphi_k) \right)}{2 - \sin(\varphi_k)} = 1$$

$$E_{sk} := \frac{1}{\gamma_k} \left[ 1 - \frac{R_f \left( 1 - \sin(\varphi_k) \right) \left( \sigma_1 - \sigma_3 \right)}{2 \cdot \sigma_3 \cdot \sin(\varphi_k)} \right]^2 \cdot k_v \cdot P_a \left( \frac{\sigma_3}{P_a} \right)^{1-\beta} = 40\, \text{MPa}$$

The value for the characteristic tangent modulus in the extern mathCAD-sheet is presented below:

$$E_{sk,\text{REFERENCE}} := 40\, \text{MPa}$$

The difference between the two is calculated:

$$\eta := \frac{E_{sk}}{E_{sk,\text{REFERENCE}}} = 1$$

The verification of the extern mathCAD-sheet obtained from Sweco obtains the same value for the tangent modulus as the verification mathCAD-sheet.
Prerequisites for design

- Regulatory design documents
  - TRVK Bro 11, TRVR Bro 11 and TK Geo 11
  - TRVFS 2001: 12
  - Eurocode 3: Steel structures - SS-EN 1993-1-(1 - 12)
  - Loads according to Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges

The design of Björnbo Bridge is based on preliminary drawing and drawings of the profile VT0.5. Equations taken from report 112 are numbered with brackets [ ] with same numbering as from report 112. Index "d" stands for "design" and index "k" stands for "characteristic"

NOTE:
The design presented in this appendix is a limited version of the full design principle according to report 112. Thus, this appendix is only presenting most important design results for Björnbo Bridge.

The SSCB should also be designed for a tamper machine for track maintenance. This has been performed for Björnbo Bridge but is not presented in this appendix.

The bridge service life is set to:
life time := 80 yr

Partial coefficients

Safetyclass of structure

\[ \gamma_d := \begin{cases} 0.83 & \text{if Safetyclass} = 1 \Rightarrow 1 \\ 0.91 & \text{if Safetyclass} = 2 \\ 1.0 & \text{if Safetyclass} = 3 \end{cases} \]

Material factors

Resistance for cross-section without regarding classification of cross-section:
\[ \gamma_{M0} := 1.0 \]

Resistance with regard to instability:
\[ \gamma_{M1} := 1.0 \]

Resistance for cross-section with regard to tensile failure:
\[ \gamma_{M2} := 1.2 \]

Soil material according to technical description for Skutskär bridge:
\[ \gamma_{Msoil} := 1.3 \]

Fatigue

The \( \gamma \)-factor is chosen with respect to safe lifetime of the structure and a severe failure mode according to SS-EN 1993-1-9: Table 3.1.

\[ \gamma_{MF} := 1.35 \]
### Profile - Skutskär Bridge

#### Corrugation details - MP200

- **Design thickness steel plate, no corrosive action is allowed to take place.**
- **Assumed to have double epoxi coating on the steel plate.**
- **Wavelength of corrugation**
- **Corrugation radius**
- **Height corrugation**
- **Angle of corrugation, Table B1.1 report 112.**
- **Tangential distance of corrugated plate, Table B1.1 report 112**
- **Radius at steel centre [b1.b]**
- **Centre of gravity corrugation [b1.d]**

#### Geometry of VT0.5

- **D**: 2.528 m + h = 2.58 m
- **L₀**: 2·D = 5.17 m
- **H₁**: 2.394 m
- **H**: 1.260 m
- **Rₜ**: 1.289 m
- **Rₜ**: 2.864 m
- **Rₖ**: 0.967 m
- **hₚ**: 50 mm

#### Steel material

- Steel quality is set to S355 for the corrugation type MP200 with corresponding values as below.

#### Characteristic yield strength steel

- **fₚ**: 355-MPa

#### Characteristic failure capacity

- **fₚ**: 430-MPa
Surrounding environment Skutskär Bridge

\[ h_{b,spj} := 0.10 \text{ m} \]

Track adjustment suggested by TRVK Bro in longitudinal direction.

\[ h_{sl} := 0.22 \text{ m} \]

Height railway sleepers

\[ h_{BE} := 10.042 \text{ m} \]

Design foundation height for pipe, bottom edge pipe, at crossing of railway track

\[ RÖK := 14.283 \text{ m} \]

Design height at top of railway track at crossing of railway track located by design foundation height.

\[ h_{rail} := 0.172 \text{ m} \]

Height railway track

\[ h_{TE} := h_{BE} + H_t + 2 \cdot h = 12.55 \text{ m} \]

Height of top edge of pipe

\[ h_c := RÖK - h_{TE} - h_{rail} = 1.57 \text{ m} \]

Soil cover depth

\[ h_{c,rest} := h_c - h_{sl} = 1.35 \text{ m} \]

Soil cover depth from top corrugation to bottom edge of railway sleeper

\[ h_{c,max} := h_{c,rest} + h_{b,spj} = 1.45 \text{ m} \]

Soil cover depth with positive track adjustment, maximum distance in-between top corrugation and bottom edge railway sleeper.

\[ h_{c,min} := h_{c,rest} - h_{b,spj} = 1.25 \text{ m} \]

Soil cover depth with negative track adjustment, minimum distance in-between top corrugation and bottom edge railway sleeper.

**Cross-section constants**

**Moment of inertia [b1.e]**

\[
I := 3 \cdot r \cdot \left( \frac{\alpha + \sin(2 \cdot \alpha)}{2} - \frac{2 \cdot \sin(\alpha)^2}{\alpha} \right) + 4 \cdot r \cdot t \cdot \left( \frac{h}{2} - e_c \right)^2 + \frac{2}{12} \cdot \frac{t}{\sin(\alpha)} \cdot (m_t \cdot \sin(\alpha))^3 = 2659 \text{ mm}^4
\]

**Plastic section modulus [b1.f]**

\[
Z := 4 \cdot \alpha \cdot r \cdot t \cdot \left( \frac{h}{2} - e_c \right) + \frac{1}{2} \cdot \frac{t}{\sin(\alpha)} \cdot (m_t \cdot \sin(\alpha))^2 = 126 \text{ mm}^3
\]

**Elastic section modulus [b1.g]**

\[
W_{el} := \frac{2 \cdot I}{h + t} = 87.2 \text{ mm}^3
\]

**Area of the steel plate [b1.c]**

\[
A := \frac{4 \cdot \alpha \cdot r \cdot t + 2 \cdot m_t \cdot t}{c} = 7.09 \text{ mm}^2
\]
Engineered backfill

<table>
<thead>
<tr>
<th>Fill material</th>
<th>Optimum density (kN/m³)</th>
<th>Density (kN/m³)</th>
<th>Angle of friction (°)</th>
<th>Static soil pressure, $k_s$</th>
<th>$C_u$</th>
<th>$d_{soil}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crushed rock</td>
<td>19.6</td>
<td>19</td>
<td>45</td>
<td>0.59</td>
<td>15</td>
<td>70</td>
</tr>
<tr>
<td>Base course material</td>
<td>20.6</td>
<td>20</td>
<td>40</td>
<td>0.36</td>
<td>10</td>
<td>70</td>
</tr>
<tr>
<td>Sub-base material</td>
<td>21.7</td>
<td>21</td>
<td>43</td>
<td>0.33</td>
<td>15</td>
<td>10</td>
</tr>
</tbody>
</table>

Table B2.1, report 112

In the report, Figure 3.1, a description of each zone can be found.

Description:

- $\rho_4$ - Unit weight of the soil up to the crown
- $\rho_{mean}$ - Mean value of the unit weight for soil material above the crown
- $\rho_{opt}$ - Optimal density according to Standard Proctor
- $\rho_s$ - Compact density for soil material in the engineered backfill, set as suggested from report 112

Tangent modulus - Method B

Factors suggested by report 112:

Equation $[b2h]$

\[
R_f = R_f\left(1 - \sin(\varphi)\right)^\beta = 0.7
\]

\[
k_v = \frac{\sin(\varphi)}{2 - \sin(\varphi)} = 0.82
\]

$P_a = 100 \text{ kPa}$

Material density $\rho$, $[b2.a]$

$\rho = 19.98 \text{ kN/m}^3$

Void ratio $e'$, $[b2.b]$

$e' = 0.3$

Modulus ratio $m'$, $[b2.c]$

$m' = 1430$

Stress exponent $\beta$, $[b2.e]$

$\beta = 0.89$

Characteristic friction angle $\varphi_k = 41$ $\text{deg}$

Design friction angle $[4.d]$

$\varphi_d = 34$ $\text{deg}$

Vertical stress $\sigma_1 = 37.47$ $\text{kPa}$

Horizontal stress $\sigma_3 = 12.94$ $\text{kPa}$

Tangent modulus $E_{sd}$, $[b2.g]$

\[
E_{ks} = 1.0 \left[ \frac{-R_f(1 - \sin(\varphi)) (\sigma_1 - \sigma_3)}{2 \cdot \sigma_3 \sin(\varphi)} \right]^{2} \cdot k_v \cdot m' \cdot \frac{\sigma_3}{P_a} \left( \frac{\sigma_3}{P_a} \right)^{1-\beta} = 40 \text{ MPa}
\]

Characteristic tangent modulus

\[
E_{sd} = \frac{1}{\gamma d \cdot \gamma_{Ms}} \left[ \frac{-R_f(1 - \sin(\varphi)) (\sigma_1 - \sigma_3)}{2 \cdot \sigma_3 \sin(\varphi)} \right]^{2} \cdot k_v \cdot m' \cdot \frac{\sigma_3}{P_a} \left( \frac{\sigma_3}{P_a} \right)^{1-\beta} = 31 \text{ MPa}
\]

Design tangent modulus
Combined stiffness of the steel pipe and the soil [4,p]

Combined characteristic stiffness ratio

\[ \lambda_{ck} := \frac{E_k D^2}{\rho I E_s} = 1230 \]

Combined design stiffness ratio

\[ \lambda_{kd} := \frac{E_k D^2}{\rho I E_s} = 946 \]

Effective soil cover depth [b3,b]

\[ \delta_{crown} := 0.013 \left( \rho V D^2 \left( \frac{H}{D} \right)^{2.8} \lambda_{kd} \left( 0.56 - 0.2 \ln \left( \frac{H}{D} \right) \right) \right) = 0.969 \text{mm} \]

Which gives a reduced soil cover depth as:

\[ h_{c,\text{red.min}} := h_{c,\text{min}} - \delta_{crown} = 1.244 \text{m} \]

Reduced soil cover depth with negative track adjustment

\[ h_{c,\text{red.max}} := h_{c,\text{max}} - \delta_{crown} = 1.444 \text{m} \]

Reduced soil cover depth with positive track adjustment

Figure Equivalent line load LM71, Report 112 - Appendix 6

\[ h_d := \max(h_{c,\text{red.max}}, h_{c,\text{red.min}}) = 1.44 \text{m} \]

Design soil cover depth Skutskär Bridge

 Loads from the train traffic:

Singletrack \( h_{c,\text{red.max}} \) = 96.83 kN/m

Train load at the maximum soil cover depth that can occur

Singletrack \( h_{c,\text{red.min}} \) = 85.37 kN/m

Train load at the minimum soil cover depth that can occur.

Design load for Skutskär Bridge:

Singletrack \( d := \max(\text{Singletrack}(h_{c,\text{red.max}}), \text{Singletrack}(h_{c,\text{red.min}})) = 96.83 \text{ kN/m} \)

Dynamic analysis according to Eurocode SS-EN 1991-2: 2003

NOTE: According to flowchart in figure 6.9 SS-EN 1991-2:2003, it is assumed that the SSCB cannot be defined as a continuos bridge and therefore the verifications in figure 6.10 is performed. Equation (2.7) and (2.8) from the report is used to construct the graph below.
The graph presents the possible frequencies, i.e. frequency band, that can occur for the Skutskär bridge according to methods provided from Eurocode.

The dynamic analysis for Björnbo Bridge is here performed without analysis of the eigenmodes, hence only the DAF is calculated according to Eurocode to multiply with the load. The DAF is not reduced as the Eurocode allows since for the Björnbo Bridge there is two railway tracks located at the railway embankment.

$$\phi_2 := \frac{1.44}{\sqrt{\frac{L_{\phi}}{m} - 0.2}} + 0.82 = 1.51$$

DAF for carefully maintained track according to SS-EN 1991-2:(2003); 6.4.5.1. (2.9)

Dynamic factor suggested from report 1112 is calculated as [3.a]:

$$r_d := \begin{cases} 
1 & \text{if } h_{c,\text{red.min}} < 2 \cdot m \\
1.10 - 0.05 \frac{h_{c,\text{red.min}}}{m} & \text{if } 2m < h_{c,\text{red.min}} < 6m \\
0.8 & \text{otherwise}
\end{cases}$$

Design for railway traffic

The characteristic load for LM71 train is calculated as:

$$N_{LM71} = R_r \phi_2 \cdot \text{Singletrack}_d \cdot \alpha_{\text{train}}$$

$$\alpha_{\text{train}} = 1.33$$

The factor $R_r$ consider a reduction of the forces in the pipe-walls when the soil cover depth is large according to equation [4.1’’- 4.1’’’’] in report 112. The equation is used below:

$$R_r := \begin{cases} 
1 & \text{if } \frac{h_{c,\text{red.min}}}{D} \leq 0.25 \\
0.77 & \text{if } 0.25 < \frac{h_{c,\text{red.min}}}{D} \leq 0.75 \\
0.5 & \text{otherwise}
\end{cases}$$
Maximum characteristic normal force:

$$N_{LM71} := R_c \phi_2 \cdot \text{Singletrackd} \cdot \alpha_{\text{train}} = 149.89 \text{kN/m}$$

Maximum normal force for fatigue calculations is based on suggestions in SS-EN 1993-2; 9.2.3., calculated as:

$$N_{LM71,\text{fat}} := R_c \cdot \text{Singletrackd} \cdot \phi_2 = 112.7 \text{kN/m}$$

### Load combination:

The load combinations for ULS and SLS is performed according to table A.2(B)S, TRVFS 2011:12, with method 2 according to SS-EN 1990, A2.3.1(5). Index "t" stands for "traffic" and index "s" stands for "soil".

- Safety class = 3
- $\gamma_d = 1$
- $\gamma_{t,\text{ULS}6.10a} = 1.5$, $\gamma_{s,\text{ULS}6.10a} = 1.35$, $\gamma_{ff} = 1.0$
- $\gamma_{t,\text{ULS}6.10b} = 1.5$, $\gamma_{s,\text{ULS}6.10b} = 0.89 \cdot 1.35$, $R_s := R_t = 1.29 \text{m}$
- $\gamma_{t,\text{SLS}} = 1.0$, $\gamma_{s,\text{SLS}} = 1.0$

#### Design load combination of 6.10a and 6.10b

$$N_{d,\text{ULS}} := \max(N_{d,\text{ULS}6.10a} \cdot N_{d,\text{ULS}6.10b}) = 299.16 \text{kN/m}$$

Maximum normal force for fatigue calculations

$$\Delta N_{d,\text{fat}} := \gamma_{ff} \cdot N_{LM71,\text{fat}} \cdot \phi_2 = 112.7 \text{kN/m}$$

#### Design bending moment in ULS:

- Safety class = 3
- $\gamma_d = 1$
- $\gamma_{t,\text{ULS}6.10a} = 1.13$, $\gamma_{s,\text{ULS}6.10a} = 1.35$, $\gamma_{ff} = 1.00$
- $\gamma_{t,\text{ULS}6.10b} = 1.5$, $\gamma_{s,\text{ULS}6.10b} = 1.2$

- Bending moment from earth pressure.

$$M_{soil,d} = -0.1 \text{kNm/m}$$

#### Maximum design bending moment in ULS:

The moments that are controlled are:

1. $M_{d,\text{ULS, pos.a}}$ - Maximum bending moment with positive moment from earth pressure 6.10a.
2. $M_{d,\text{ULS, pos.b}}$ - Maximum bending moment with positive moment from earth pressure 6.10b.
3. $M_{d,\text{ULS, neg.a}}$ - Absolute value of moment with negative moment from earth pressure 6.10a.
4. $M_{d,\text{ULS, neg.b}}$ - Absolute value of moment with negative moment from earth pressure 6.10b.

$$M_{d,\text{ULS}} = \gamma_{s,\text{ULS}} \cdot M_{soil,d} + \gamma_{t,\text{ULS}6.10j} \cdot M_{t,d} \quad j = a \& b$$

$$M_{d,\text{ULS}} := \max(M_{d,\text{ULS, pos.a}} \cdot M_{d,\text{ULS, pos.b}}, 0, |M_{d,\text{ULS, neg.a}}|, |M_{d,\text{ULS, neg.b}|}) = 7.06 \text{kNm/m}$$
Design bending moment in SLS:

Safetyclass = 3 \quad \gamma_d = 1

\gamma_{t\text{SLS}} = 1.0 \quad \gamma_{s\text{SLS}} = 1.0

According to table A2.6, SS-EN1990/A1:2010(E):

\[ M_{d\text{MAX.SLS}} := \gamma_{s\text{SLS}} M_{\text{soil.d}} + \gamma_{t\text{SLS}} M_{t.d} r_d = 4.67 \text{ kNm/m} \]

\[ M_{d\text{MIN.SLS}} := \gamma_{s\text{SLS}} M_{\text{soil.d}} + \gamma_{t\text{SLS}} \frac{-M_{t.d} r_d}{2} = -2.48 \text{ kNm/m} \]

\[ M_{d\text{SLS}} := \max(M_{d\text{MAX.SLS}}, |M_{d\text{MIN.SLS}}|) = 4.67 \text{ kNm/m} \]

Design bending moment fatigue:

\[ \Delta M_{d\text{fat}} := \gamma_{fF} M_{t.d,\text{fat}} r_d \cdot 1.25 = 4.1 \text{ kNm/m} \]

With suggestions from supervisor the factor 1.5 suggested from report 112 is changed to 1.25.
Design verifications suggested for ULS and SLS from report 112:

The verification is made to ensure safety for a plastic hinge to develop in the pipe-wall. Stress calculated with Navier equation:

\[ \sigma_{SLS} := \frac{N_{d,SLS}}{A} + \frac{M_{d,SLS}}{W_{el}} = 83.41 \text{ MPa} \]

NOTE:
Verification against settlements in the surrounding soil is not performed in this calculation as suggested in report 112. It is assumed that the settlement is small and hence negligible.

Verification in ULS:
The verifications are performed with suggested methods from report 112 with adaptions to Eurocode 3 p.41. Eurocode 3: Design of Steel structures - SS-EN 1993-1-1.

Characteristic Normal force:  
Design Normal force:
\[ N_{c,Rk} := f_{yk} \cdot A = 2519 \text{ kN/m} \]
\[ N_{c,Rd} := \frac{f_{yk} \cdot A}{\gamma_{M0}} = 2519 \text{ kN/m} \]

Plastic section modulus:
Elastic section modulus:
\[ W_{pl} := Z = 125.74 \text{ mm}^3/\text{mm} \]
\[ W_{el} = 87.2 \text{ mm}^3/\text{mm} \]
\[ W_y := \frac{W_{pl}}{W_{el}} = 1.44 \]

Design value for bending moment according to SS-EN 1993-1-1 eq.(6.13), cross-section classification assumed to class 1 or 2.

Characteristic bending moment:
Design bending moment:
\[ M_{y,Rk} := W_{pl} f_{yk} = 44.64 \text{ kNm/m} \]
\[ M_{y,Rd} := \frac{W_{pl} f_{yk}}{\gamma_{M0}} = 44.64 \text{ kNm/m} \]

Verification for local buckling according to appendix 1, report 112, \[b1.h]\] :
\[ M_{ucr} := 1.429 - 0.156 \ln \left( \frac{\frac{f_{yk}}{227 \text{ MPa}}}{\frac{1}{2}} \right) \]
\[ M_{y,Rd} = 51.88 \text{ kNm/m} \]

\[ M_{y,Rd} := M_{ucr} \text{ if } M_{ucr} \leq M_{y,Rd} = 44.64 \text{ kNm/m} \]

Design values for normal force and bending moment about y-y axis:
\[ N_{Ed} := N_{d,ULS} = 299.16 \text{ kN/m} \]
\[ M_{y,Ed} := M_{d,ULS} = 7.06 \text{ kNm/m} \]

Verification for structural components subjected to bending of pressure with constant cross-section in SS-EN 1993-1-1 eq.(6.61) and according to report 112 with \[5.b´\] (3.17).
Verification for plastic hinge in the upper part of the pipe with \( M_{y, Ed} = 0 \text{kNm/m} \):

The verification is also performed with methods suggested in BSK07 with \( M_{y, Ed} = 0 \text{kNm/m} \) as (3.18):

\[
\alpha_c = \eta_j \cdot \chi_{y, b} = 0.42
\]

\[
\alpha_c = \begin{cases} 
\alpha_c & \text{if } \alpha_c \geq 0.8 = 0.8 \\
0.8 & \text{otherwise}
\end{cases}
\]

Verification that a plastic hinge is not developing in the bottom part of the pipe:

\[
N_{d, ULS} < N_{cr,4} \quad N_{d, ULS} = 299 \text{kN/m}
\]

\[
\xi_{Ncr,4} := 1.0 \quad \eta_{j,4} := 1.0 \quad \mu_{Ncr,4} := 1.22 \quad \text{According to report 112 p.37}
\]

\[
N_{cr,el,4} := \frac{3 \xi_{Ncr,4}}{\mu_{Ncr,4}} \frac{E_{sd} E_s 1}{\max(R_b, R_c, R_s)} = 6013 \text{kN/m} \quad [5.d]
\]

\[
N_{cr,4} := \begin{cases} 
N_{cr,el,4} & \text{if } \frac{N_{cr,el,4}}{A f_{yd}} \leq 0.5 \\
\left[f_{yd} A \left(1 - \frac{1}{4} \frac{f_{yd} A}{N_{cr,el,4}}\right)\right] & \text{otherwise}
\end{cases}
\]

\[
\text{Ver}_5 := \begin{cases} 
"OK" & \text{if } N_{d, ULS} \leq N_{cr,4} = "OK" \\
"NOTOK" & \text{otherwise}
\end{cases}
\]

Verification of bolted joints:

\[
F_{b, Rd} := \frac{k_1 \cdot \alpha_b \cdot f_{ubk} \cdot d_{bolt} \cdot t}{\gamma_M 2} = 83.86 \text{kN} \quad [5.g'] \quad (3.19)
\]

\[
F_{v, Rd} := \frac{\alpha_c \cdot f_{ubk} / A_{nom}}{\gamma_M 2} = 98 \text{kN}
\]

Bearing capacity for tensile forces for one bolt:

\[
F_{t, Rd} := \frac{0.9 \cdot f_{ubk} / A_{nom}}{\gamma_M 2} = 147 \text{kN} \quad [5.i'] \quad (3.20)
\]

\[
F_{t, Ed} := \frac{2 \cdot W_{el} \cdot f_{yd}}{\gamma_M 2 \cdot \eta_{bolt}} = 40.46 \text{kN} \quad [5.j'] \quad (3.20)
\]

Verification of that the pipe will manage lifting, assembly at place etc.:

The verification is made according to p.40 in report 112.

Shape := "low-rise"

\[
\eta_m := \begin{cases} 
\text{Shape = "circular", } 0.13 \frac{m}{\text{kN}}, 0.20 \frac{m}{\text{kN}} & \frac{\lambda_{lk}}{E_{sd} D} = 0.016 \frac{m}{\text{kN}} \\
\eta_m = 0.2 \frac{m}{\text{kN}}
\end{cases}
\]
Design fatigue Skutskär SSCB

The verification of Skutskär SSCB for fatigue is performed according to SS-EN 1993-1-9:2005(Swe). The method used is the lambda-method with the criteria that the bridge should be able to be utilized its full lifetime.

\[
\text{lifetime} = 80\text{-yr}
\]

The verification that is used is:

\[
\frac{\gamma_{ff} \Delta \sigma_{E,2}}{\Gamma \Delta \sigma_c} + \frac{\gamma_{ff} \Delta \tau_{E,2}}{\Gamma \Delta \tau_c} \leq 1.0
\]

(3.24) According to (8.3) in SS-EN 1993-1-9:2005(Swe)

Partial coefficients: Load: \( \gamma_{ff} = 1.00 \) Material: \( \gamma_{MF} = 1.35 \)

Category numbers:

\( \Delta \sigma_{e,71} := 71\text{-MPa} \)

Category number for bolted joints, steel material, according to J.3.2.2 in TRVK Bro 11

\( \Delta \sigma_{e,5} := 125\text{-MPa} \)

Category number for steel material according to table 8.1 in SS-EN 1993-1-9:2005(Swe), detail 5.

\( \Delta \sigma_{e,14} := 50\text{-MPa} \)

Category number for bolted joints in tension according to table 8.1 in SS-EN 1993-1-9:2005(Swe), detail 14.

\( \Delta \tau_{e,15} := 100\text{-MPa} \)

Category number for bolted joints in shear plane according to table 8.1 in SS-EN 1993-1-9:2005(Swe), detail 15.

Stress top surface corrugation

\[\Delta \sigma_{TS} = -\Delta N_{d,\text{fat}} A - \frac{M_{d,\text{fat}}}{W_{el}} - \frac{M_{\text{soil},d}}{W_{el}}\]

Stress bottom surface corrugation

\[\Delta \sigma_{BS} = -\Delta N_{d,\text{fat}} A + \frac{M_{d,\text{fat}}}{W_{el}} + \frac{M_{\text{soil},d}}{W_{el}}\]

Stress range in the crown:

\[\Delta \sigma := \max\left(\left|\sigma_{TS}\right|, \left|\sigma_{BS}\right|\right) = 36.45\text{-MPa}\]

Stress range at the first bolted joint below the crown:

When the first bolted joint not is located at the crown and thus has large soil cover depth, the bending moment for fatigue is reduced with the factor \((h_c/h_f)^2\). The stresses are calculated in the same manner as for the top/bottom corrugation at the crown with the reduction added.

Stress range in the crown:

\[\Delta \sigma_{hf} := \max\left(\left|\sigma_{TS,hf}\right|, \left|\sigma_{BS,hf}\right|\right) = 33.6\text{-MPa}\]
Lambda-method:
The design nominal stress range according to 6.3 in SS-EN 1993-1-9:2005(swe) and 9.5 in SS-EN 1993-2:2006(Swe) is calculated as:

\[ \Delta\sigma_{E2} = \lambda \phi_2 \Delta\sigma \]

According to (9.2) in SS-EN 1993-2:2006(Swe)

\[ \phi_2 = 1.51 \]

The un-reduced DAF according to SS-EN 1991-2:2005(Swe)

\[ \lambda = \lambda_1 \lambda_2 \lambda_3 \lambda_4 \]

But, \( \lambda < \lambda_{\text{max}} \)

\( \lambda_1 \) Stress range factor depending on length of influence line

\( \lambda_2 \) Factor considering traffic volume

\( \lambda_3 \) Factor considering the bridge lifetime

\( \lambda_4 \) Factor for structural component the is subjected by several tracks

\( \lambda_{\text{max}} \) Maximum value for the fatigue-limit. \( \lambda_{\text{max}} \neq 1.4 \) is the maximum value allowed according to SS-EN 1993-2:2006 but according to E.3.1 in TRVK Bro 11 \( \lambda \) should be used without maximum value

Lambda 1 determined from graph.

Lambda 2 determined from graph.

According to table 9.6 in SS-EN 1993-2:2006(Swe)

\[ \lambda_3 := \left( \frac{\text{lifetime}}{100 \text{ yr}} \right)^{\frac{1}{5}} = 0.96 \]

According to TRVFS 2011:12

\[ \lambda_4 := 1.0 \]

According to E.3.1 in TRVK Bro 11:

\[ \lambda := \lambda_1 \lambda_2 \lambda_3 \lambda_4 = 1.02 \]
Verification for FLS:

Steel plate material in the crown:

\[ \sigma_{\text{crown}} := \lambda \cdot \phi_2 \cdot \Delta \sigma = 56 \text{ MPa} \]

\[ \frac{\Delta \sigma_{\text{c,5}}}{\gamma_{\text{Mf}}} = 93 \text{ MPa} \]

Verification for stress in crown:

\[ \Delta F_{d, \text{fat,shear}} := \frac{\Delta N_{d, \text{fat}}}{n_{\text{bolt}}} = 7.51 \text{ kN} \]

Shear force in each bolt:

\[ \Delta \tau_{\text{E,2}} := \lambda \cdot \phi_2 \cdot \frac{\Delta F_{d, \text{fat,shear}}}{A_{\text{nom}}} = 47 \text{ MPa} \]

The shear stress in each bolt is calculated with the assumption that no corrosive action takes place.

The tensile force for fatigue in each bolt is calculated as suggested from report 112 with \([5, h]\) thus it becomes:

\[ \frac{\varepsilon_{\text{red, hf}} \cdot 2 \cdot M_{d, \text{fat}} \cdot f_{yd}}{f_{yd} \cdot \gamma_{\text{Mf}} \cdot \sigma_{\text{nom}}} = 3.56 \text{ kN} \]

\[ \varepsilon_{\text{red, hf}} = 0.93 \]

\[ \Delta F_{d, \text{fat,tens}} := \frac{\Delta F_{d, \text{fat,tens}}}{n_{\text{bolt}}} = 3.56 \text{ kN} \]

\[ \Delta \sigma_{\text{E,2,bolt}} := \lambda \cdot \phi_2 \cdot \frac{\Delta F_{d, \text{fat,tens}}}{A_{\text{nom}}} = 22 \text{ MPa} \]

Tensile stress in each bolt.
Summary of Skutskär SSCDB design

Steel quality:
\[ f_{yd} = 355-\text{MPa} \quad f_{ad} = 430-\text{MPa} \quad f_{uad} = 667-\text{MPa} \]
\[ \Delta \sigma_{c,5} = 125-\text{MPa} \quad \Delta \sigma_{c,71} = 71-\text{MPa} \quad \Delta \sigma_{c,14} = 50-\text{MPa} \quad \Delta \tau_{c,15} = 100-\text{MPa} \]

Tangent modulus for soil:
Design modulus: \( E_{sd} = 31-\text{MPa} \)
Characteristic modulus: \( E_{sk} = 40-\text{MPa} \)

SSCB Skutskär:
- Design combined stiffness ratio: \( \lambda_{fd} = 946 \)
- Characteristic combined stiffness ratio: \( \lambda_{fk} = 1230 \)
- Corrugation: \( c = 200-\text{mm} \quad h = 55-\text{mm} \)
- Thickness plate: \( t = 6.00-\text{mm} \)
- Area plate: \( A = 7.09-\text{mm}^2 \)
- Moment of inertia: \( I = 2659-\text{mm}^4 \)
- Elastic section modulus: \( W_{el} = 87.2-\text{mm}^3 \)
- Plastic section modulus: \( W_{pl} = 126-\text{mm}^3 \)

Theoretical soil cover depth:
- \( h_{c,\text{min}} = 1.245 \text{ m} \)
- \( h_{c,\text{max}} = 1.445 \text{ m} \)

Design soil cover depth: \( h_d = 1.444 \text{ m} \)
Estimated crown rise: \( \delta_{\text{crown}} = 0.969-\text{mm} \)

Loads:
- Design normal force from 6.10a & 6.10b: \( N_{d,ULS} = 299.16-\text{kN/m} \)
- Elastic buckling force: \( N_{cr,el} = 2726-\text{kN/m} \)
- Design moment from earth pressure: \( M_{soil,d} = -0.1-\text{kNm/m} \)
- Design shear capacity bolt for failure of the bolt: \( F_{v,Rd} = 98-\text{kN} \)
- Design shear capacity for failure of the steel plate: \( F_{b,Rd} = 83.86-\text{kN} \)
- Design bending moment for ULS: \( M_{d,ULS} = 7.06-\text{kNm/m} \)
- Design tensile capacity for bolt: \( F_{t,Rd} = 147-\text{kN} \)
- Design bending moment for SLS: \( M_{d,ULS} = 4.67-\text{kNm/m} \)
- Design tensile force in bolt: \( F_{t,Ed} = 40.46-\text{kN} \)
- Design normal force fatigue: \( \Delta N_{d,\text{fat}} = 112.7-\text{kN/m} \)
- Stress at crown: \( \Delta \sigma = 36.45-\text{MPa} \)
- Design bending moment fatigue: \( \Delta M_{d,\text{fat}} = 4.1-\text{kNm/m} \)
- Stress at depth of first bolted joint: \( \Delta \sigma_{hf} = 33.6-\text{MPa} \)

Buckling force: \( N_{cr} = 1937-\text{kN/m} \)
Utilization rates:

Plastic hinge developing in pipe wall (SLS):

\[ \text{UR}_1 := \frac{\sigma_{\text{SLS}}}{f_y} = 23\% \]

Plastic hinge developing at crown, only N_{Ed} Eurocode (ULS):

\[ \text{UR}_3 := \frac{N_{Ed}}{X_Y b N_{c,Rk}} + k_{yy} \left( \frac{M_{y,Rk}}{\gamma_{M1}} \right) = 16\% \]

Plastic hinge developing at bottom part of pipe:

\[ \text{UR}_5 := \frac{N_{d,ULS}}{N_{cr.4}} = 13\% \]

Bolts in tension (ULS):

\[ \text{UR}_7 := \left( \frac{W_{Ed f_{Ed}}}{a \cdot \eta_{\text{bolt}} \cdot F_{t,Rd}} \right) = 33\% \]

Pipe during assembly (SLS):

\[ \text{UR}_9 := \frac{X_{Ed} D}{E_{\text{md}} m} = 6\% \]

Stress in steel material adjacent to a bolted joint (FLS):

\[ \text{UR}_{11} := \frac{\gamma_{f_f} \sigma_{\text{bolt}}}{\Delta \sigma_{c.71}} = 99\% \]

Bolts subjected by tensile stresses (FLS):

\[ \text{UR}_{13} := \frac{\gamma_{f_f} \Delta \sigma_{E.2,\text{bolt}}}{\Delta \sigma_{c.14}} = 61\% \]

Bolts in interaction (Shear and tension) (FLS):

\[ \text{UR}_{14} := \left( \frac{\gamma_{f_f} \Delta \sigma_{E.2,\text{bolt}}}{\Delta \sigma_{c.14}} \right)^3 + \left( \frac{\gamma_{f_f} \Delta \tau_{E.2}}{\Delta \tau_{c.15}} \right)^5 = 33\% \]

Plastic hinge developing at crown (ULS):

\[ \text{UR}_2 := \frac{N_{Ed}}{X_Y b N_{c,Rk}} + k_{yy} \left( \frac{M_{y,Rk}}{\gamma_{M1}} \right) = 33\% \]

Plastic hinge developing at crown, only N_{Ed} BSK07 (ULS):

\[ \text{UR}_4 := \left( \frac{N_{Ed}}{X_Y b N_{c,Rk}} \right) + \left( \frac{0.0 \text{ kNm}}{M_{y,Rk}} \right) = 23\% \]

Bolts in shear (ULS):

\[ \text{UR}_6 := \frac{N_{Ed}}{n_{\text{bolt}} F_{v,Rd} + n_{\text{bolt}} F_{t,Rd}} = 24\% \]

Bolts in interaction (shear and tension) (ULS):

\[ \text{UR}_8 := \frac{N_{Ed}}{n_{\text{bolt}} F_{v,Rd} + 1.4 F_{t,Rd}} = 40\% \]

Stress at crown in steel material (FLS):

\[ \text{UR}_{10} := \frac{\gamma_{f_f} \sigma_{\text{crown}}}{\Delta \sigma_{c.5}} = 61\% \]

Bolts subjected by shear stresses and steel material for shear (FLS):

\[ \text{UR}_{12} := \frac{\gamma_{f_f} \Delta \tau_{E.2}}{\Delta \tau_{c.15}} = \left( \frac{64}{15} \right) \% \]

Bolts in tension (ULS):

\[ \text{UR}_7 := \frac{N_{Ed}}{n_{\text{bolt}} F_{v,Rd} + 1.4 F_{t,Rd}} = 40\% \]
Appendix B

Parametric studies – Static load case

The parametric studies performed according to methods prescribed in report 112 were performed for the configurations described in chapter 5.1. The results presented here are also briefly commented since the parametric study only was performed to ensure that the configurations used to analyse for dynamic approaches also fulfil the static approaches. The steel thickness for the SSCB was set to $t = 6.0$ mm to follow the design thickness set for the Björnbo Bridge. Thus, some of the verifications present results that are not accepted in a real design situation.
Soil cover depth

The compiled result for different soil cover depth combined with the suggested profile for Björnbo Bridge, i.e. Viacon profile VT0.5, are presented for ULS and SLS in Figure B.1. The calculated utilization rates present low utilization rates and low variation with linear trends. Since almost no variation is seen in the utilization rates and that the different configurations are all below their maximum capacity with different soil cover depth for the suggested profile it can be concluded that the VT0.5-profile will be able to sustain loads with increased soil cover depth.

In Figure B.2, the compiled calculated result regarding FLS for increased soil cover depth and the VT0.5-profile is presented. Almost all configurations that are evaluated will be able to survive the full design life set for the SSCB, i.e. 80 years. The configurations with low soil cover depth, less than \( \approx 1.5 \text{m} \), will suffer from fatigue problems located in the steel material near bolted joints.

The stress levels adjacent to the VT0.5-pipe, characteristic tangent modulus and combined stiffness ratio are presented in Figure B.3. The stress levels show linear increasing trends whilst the characteristic tangent modulus has little variation for different soil cover depths together with the VT0.5-profile. In addition, the combined stiffness ratio of the VT0.5-profile and the soil cover depth shows an increasing trend with increasing soil cover depth.

![Figure B.1: Utilization rates for increased soil cover depth regarding ULS and SLS.](image-url)
PARAMETRIC STUDIES – STATIC LOAD CASE

Figure B.2: Utilization rates for increased soil cover depth regarding FLS.

Figure B.3: Stress levels adjacent to the SSCB-profile, characteristic tangent modulus and combined stiffness ratio for different soil cover depths.
VT-profiles

The compiled results for different configurations of VT-profiles for ULS and SLS are presented in Figure B.4, the utilization rates that is presented show small variation and linear trends for increasing of SSCB-profile-dimensions regarding VT-profiles. As can also be seen, all configurations are below their maximum capacity regarding ULS and SLS.

The result obtained regarding FLS for different configurations for VT-profiles are presented in Figure B.5. The variation seems to follows linear trends and for some cases the variation can be quite large, the verification for interaction of shear and tension for bolted joints seem to vary depending on the SSCB-profile dimension. For the FLS verifications, it can be seen that for the configurations the utilization rates are higher and thus it can be concluded that FLS is the design state that set the final dimensions for the VT profiles. For one of the verifications, i.e. steel material near a bolted joint, all configurations will suffer from fatigue during the SSCB design lifetime.

Calculated results for the surrounding environment adjacent to the SSCB-profile and calculated result that regard the combined stiffness ratio for different SSCB-profile-dimensions are presented in Figure B.6. It can be seen that the vertical and horizontal stress shows linear increasing trends while the tangent modulus has small variations for different SSCB-profile configurations. In addition, the combined stiffness ratio is highly affected for different SSCB-profile dimensions according and thus for larger VT-profiles a higher combined stiffness is attained.

Figure B.4: VT-profiles utilization rates for ULS and SLS.
PARAMETRIC STUDIES – STATIC LOAD CASE

Figure B.5: VT-profiles utilization rates for FLS.

Figure B.6: Stress levels adjacent to increasing VT-dimensions, characteristic tangent modulus and combined stiffness ratio for VT-profiles.

VE-profiles

The compiled results for utilization rates for increasing VE-profiles regarding ULS and SLS are presented in Figure B.7, the calculated result shows small variation in the utilization of the material.
VE-profiles analysed for increasing SSCB-profile-dimensions in FLS are presented in Figure B.8. The trends for the verifications seems to be linear for the utilization rates. Here, some larger variations can be seen for the interaction of shear and tension in bolted joints whilst again the verification for the steel material adjacent to a bolted joint does not fulfill the design criteria.

In Figure B.9, the stress level adjacent to the VE-profile, characteristic tangent modulus and combined stiffness ratio for the VE-profiles are presented. It can here be noted that the combined stiffness ratio is lower than for the VT-profile by comparing with values in Figure B.6. The vertical stress levels show an increasing trend, whilst the horizontal stress levels seem to flatten out for larger VE-profiles. The characteristic tangent modulus show small variation.

*Figure B.7: VE-profiles utilization rates for ULS and SLS.*
PARAMETRIC STUDIES – STATIC LOAD CASE

Figure B.8: VE-profiles utilization rates for FLS.

Figure B.9: Stress levels adjacent to increasing VE-profiles, characteristic tangent modulus and combined stiffness ratio for VE-profiles.

Summary
The parametric studies with methods from report 112 shows that all configurations that will be analysed in finite element approximations will be able to sustain loads regarding ULS and SLS criterions. Thus, the conclusion for utilization rates regarding
ULS and SLS are that the material is poor utilized and thus the profile dimensions is overestimated. However, there will be for some configurations that the FLS criterions will not be fulfilled, which gives that FLS is the governing design state to be followed. Moreover, it should be remembered that some parameters are fixed to the design situation and to confine the analysis performed to only a small set of parameter variation. Thus, if the parameter would be able to change, different results would be obtained and the SSCB would be able to be designed without suffering from fatigue.