Low-cycle Fatigue of Steel Piles in Integral Abutment Bridges

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“The only good joint is no joint”

Henry Derthick, former Engineer of Structures at Tennessee Department of Transportation.
Preface

After five years of university studies it is finally time to cross the finishing line. It has been a long and winding road, but nevertheless it has been a pleasant journey. Five years ago, I started my studies towards a degree in business administration. But somewhere along the road I found an interesting path, and I could not resist the temptation to find out where that path would take me. So I made a turn, leaving annual reports, accounts, and key ratios behind me. And I have never regretted that choice. I found a more interesting area and became interested in such weird stuffs like concrete, steel, and bridges. So, this journey ends in a master degree in civil engineering, instead of economics. The master’s thesis is the final document of this journey, but there are still several other journeys to be made and areas to be explored, and you just never know what path your life is going to take.

This report has been initiated by Ramböll Sverige AB as a part of a SBUF-project about integral abutment bridges. This project is carried out together with LTU, NCC and Ramböll.

I would like to thank the people who made this work come true.

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Robert Hällmark
Abstract

Besides the safety aspects, the economy is the single most important factor when bridges are designed. Lowering the life cycle cost of bridges means that less tax-money would be spent, and that should be in the interest of the general public. Today, bridges in Sweden are generally designed with movable joints and bearings. Leaking joints are a major reason to corrosion problems, and it would be preferable if bridges were designed without these. Integral abutment bridges are bridges without any movable joints. The superstructures are rigidly connected to the abutments, which generally are supported by a single row of flexible piles. The largest benefits of integral abutment bridges are the lower construction- and maintenance costs.

Movable joints and bearings are used in order to handle the expansion and contraction of the superstructure due to temperature changes. If these components are not used, then additional forces will be transferred to the abutments. Therefore, abutments in integral bridges will be laterally displaced as the temperature changes. The top of the piles will also be displaced and forces as well as moments will be induced in the piles. Pile stresses can locally exceed the yield strength of the pile material and plastic hinges can be developed. The development of plastic hinges in steel piles is allowed in the design of integral bridges in some states in the USA. The Swedish National Road Administration seems to be more hesitant about allowing pile stresses above the yield strength. And there seems to be a concern about whether or not there could be problems with fatigue involving plastic deformations, low-cycle fatigue. The aim of this thesis is to answer if, how and when low-cycle fatigue failures might happen in piles supporting integral abutment bridges.

First of all, a literature review has been done in order to get a better understanding of the problem and to gain knowledge about the research areas that are involved in this report. Integral bridges have been studied in general and especially their thermal behaviour. Other areas that have been studied are piles, fatigue, effective bridge temperature, traffic loads, and the Monte Carlo method.

In order to simulate pile strains in integral abutment bridges, a temperature simulation model and a traffic simulation model were created. One example bridge, Leduån Bridge, has been used in the calculations throughout the report. It is a single span composite road bridge with a span length of 40 m. A couple of input parameters have been varied in order to find out in which amount they influence the pile fatigue. Some of the varied parameters are the lateral soil stiffness, pile cross-section, the location of the bridge (different climates), and the length of the bridge.

The temperature model is based on shade air temperature measurements during 30 years at five locations in Sweden. These temperatures are transformed into Effective Bridge Temperatures (EBT) in order to simulate the lateral displacements of the abutments. The seasonal temperature changes will give an annual strain cycle in the piles, and there will also be daily temperature variations giving smaller strain cycles. Variations in the vertical temperature gradient in the superstructure are also taken into consideration, since these will give rotations of the top of the piles as well as small lateral displacements.
The traffic model is based on vehicle gross weights from BWIM measurements performed by the Swedish National Road Administration. Two traffic models have been used. The first one is based on the traffic intensity and gross weights at the road E22, and the other one is based on measurements from National Road 67.

The traffic load model has been combined with the temperature model, and Monte Carlo simulations of pile strains have been performed. The simulation results can be presented as pile strain spectra, involving cycles with periods from seconds up to years. A load spectrum during the designed lifetime of the bridge, 120 years, would involve more than 50 million strain cycles. These cycles have to be identified and counted in order to perform cumulative fatigue calculations. A method called the Rain-flow method has been used to identify the cycles, count them and sort them.

The results from the calculations in this report indicate that low-cycle fatigue failures are not expected in piles supporting integral abutment bridges, at least up to a bridge length of 100 m. The calculation model is rather conservative and it is possible that even longer bridges can be constructed without problems with low-cycle fatigue. The importance of lowering the lateral soil stiffness can also be studied in the results. This fact has been noted in several other studies as well. It is also noted that some pile cross-sections seem to be more suitable than others for integral abutment bridges.
Sammanfattning (Summary in Swedish)


Rörelsefogar och lager används i konventionella broar för att hantera variationer i överbyggnadens längd, på grund av varierande temperatur. Om broar byggs utan dessa två komponenter innebär det att extra tvångskrafter kommer att verka på landfästena och deras pålar. Landfästena hos broar med integrerade landfästen kommer därför att utsättas för horisontella förskjutningar då temperaturen i överbyggnaden varierar. Påltopparna som är ingjutna i landfästena kommer också att förskjutas till följd av landfästenas rörelse, samt påverkas av såväl moment som skjuvkrafter. Spännningarna i pålarna kan lokalt överskrida flytgrensen och flyteläder uppstå. I vissa delstater i USA konstruerar man broar där flyteläder i pålarna tillåts i bruksgränsstillstånd. Vägverket i Sverige verkar vara mer tvivlande inställda till att tillåta flyteläder i bruksgränsstillstånd. Ett av orosmomenten är huruvida låg-cyklisk utmattning kan komma att bli ett problem om flyteläder tillåts. Syftet med denna rapport är att föröka svara på frågorna om, hur och när låg-cykliska utmattningsbrott kan komma att inträffa i pålar i integrerade landfästen.

Examensarbetet inleddes med en litteraturstudie för att få en djupare inblick i de aktuella frågeställningarna, samt för att inhämta kunskap från de olika forskningsområden som berör detta examensarbete. Broar med integrerade landfästen har studerats i allmänhet och i synnerhet deras temperaturrörelser. Vidare så har även följande områden omfattats av studien: pålar, utmattning, effektiv brotemperatur, trafiklast samt Monte Carlo metoden.

En temperaturmodell samt en trafiklastmodell har tagits fram för att kunna simulera de tänkta pålarna i integrerade landfästen kan komma att utsättas för. Alla beräkningar i denna rapport är gjorda med en bro som utgångspunkt. Den bro som används vid beräkningarna är en 40 m lång vägbro över Leduån utanför Nordmaling. Bron är en tvåfogad samverksbro med en fri brobredd på 5 m. För att kunna studera hur olika faktorer påverkar utmattningen har en rad parametrar varierats, bland annat följande: den horisontella jordstylvheten, pålarnas tvärsnitt, klimatets påverkan (olika placeringsorter) samt längden på bron.
Sammanfattning

Temperaturmodellen som används i denna rapport är baserad på trettio års mätningar av lufttemperaturen på fem platser runt om i Sverige. Lufttemperaturerna omräknas till effektiv brotemperatur (EBT) för att kunna utföra en simulering av de horisontella förskjutningarna av landfästena. De årliga temperaturvariationerna kommer att ge upphov till en årlig töjningscykel i pålarna. De dagliga temperaturvariationerna kommer också att bidra med töjningscyklar men med lägre amplituder. Hansyn har också tagits till att den vertikala temperaturgradienten i överbyggnaden varierar, eftersom dessa variationer kommer ge upphov till rotationer av pätopparna såväl som små horisontella förskjutningar av desamma.


Trafikmodellen har länkats samman med temperaturmodellen och en Monte Carlo simulering av pålarnas töjning har utförts. Resultatet av simuleringarna kan åskådliggöras som ett lastspektra innehållande cyklar med perioder från sekunder upp till år. Ett lastspektrum under brons tekniska livslängd, 120 år, skulle innehålla mer än 50 miljoner töjningscykler. Dessa cykler måste kunna identifieras, grupperas och summeras för att en kumulativ utmattningsberäkning skall kunna utföras. En metod vid namn ”Rainflow”-metoden har använts för att kunna identifiera cyklerna och sammanställa dem.

Resultatet från beräkningarna i denna rapport tyder på att utmattningsbrott av låg-cyklisk karaktär ej kommer att ske, åtminstone inte så länge som brons längd ej överskrider 100 m. Beräkningsmodellen är ganska konservativ och det är fullt möjligt att även längre broar kan byggas utan några som helst problem med låg-cyklisk utmattning. Vidare kan det från resultaten utläsas att den horisontella jordstyvhetens väldigt stort inflytande på töjningarna, vilket även noterats i flertalet andra studier. Det förefaller också som så att en del påtvärsnitt är mer lämpade än andra för användning i integrerade landfästen.
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## Nomenclature

### Greek letters

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<tr>
<th>Symbol</th>
<th>Description</th>
<th>Units</th>
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<tr>
<td>( \alpha )</td>
<td>Thermal coefficient of a material</td>
<td>( ^\circ \text{C}^{-1} )</td>
</tr>
<tr>
<td>( \beta )</td>
<td>Constant</td>
<td>[-]</td>
</tr>
<tr>
<td>( \Delta_{\text{abut}} )</td>
<td>Lateral abutment displacement</td>
<td>[m]</td>
</tr>
<tr>
<td>( \Delta \varepsilon )</td>
<td>Strain range</td>
<td>[-]</td>
</tr>
<tr>
<td>( \Delta L_b )</td>
<td>Changes in bridge length</td>
<td>[m]</td>
</tr>
<tr>
<td>( \Delta T_{\text{E}} )</td>
<td>Non-linear temperature distribution</td>
<td>( ^\circ \text{C} )</td>
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<td>( \Delta T_M )</td>
<td>Linearly varying temperature component</td>
<td>( ^\circ \text{C} )</td>
</tr>
<tr>
<td>( \Delta T_N )</td>
<td>Uniform temperature component</td>
<td>( ^\circ \text{C} )</td>
</tr>
<tr>
<td>( \Delta T_{\text{sol}} )</td>
<td>Uniform temperature change from direct solar radiation</td>
<td>( ^\circ \text{C} )</td>
</tr>
<tr>
<td>( \varepsilon )</td>
<td>Strain</td>
<td>[-]</td>
</tr>
<tr>
<td>( \varepsilon_a )</td>
<td>Stain amplitude</td>
<td>[-]</td>
</tr>
<tr>
<td>( \varepsilon_{cs} )</td>
<td>Final concrete shrinkage</td>
<td>[-]</td>
</tr>
<tr>
<td>( \varepsilon_f )</td>
<td>Elongation at fracture</td>
<td>[-]</td>
</tr>
<tr>
<td>( \varepsilon'_f )</td>
<td>Fatigue ductility coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>( \varepsilon_y )</td>
<td>Yield strain</td>
<td>[-]</td>
</tr>
<tr>
<td>( \phi )</td>
<td>Soil friction angle</td>
<td>( ^\circ )</td>
</tr>
<tr>
<td>( \gamma )</td>
<td>Unit weight of a material</td>
<td>[N/m(^3)]</td>
</tr>
<tr>
<td>( \varphi )</td>
<td>Load factor</td>
<td>[-]</td>
</tr>
<tr>
<td>( \mu )</td>
<td>Mean value</td>
<td>[-]</td>
</tr>
<tr>
<td>( \theta )</td>
<td>Rotation angle</td>
<td>[-]</td>
</tr>
<tr>
<td>( \theta_i )</td>
<td>Coefficient of inelastic rotation capacity</td>
<td>[-]</td>
</tr>
<tr>
<td>( \sigma )</td>
<td>Standard deviation</td>
<td>[-]</td>
</tr>
<tr>
<td>( \sigma )</td>
<td>Stress</td>
<td>[N/m(^2)]</td>
</tr>
<tr>
<td>( \sigma_u )</td>
<td>Ultimate tensile strength</td>
<td>[N/m(^2)]</td>
</tr>
<tr>
<td>( \sigma'_f )</td>
<td>Fatigue strength coefficient</td>
<td>[N/m(^2)]</td>
</tr>
<tr>
<td>( \psi )</td>
<td>Reduction factor</td>
<td>[-]</td>
</tr>
<tr>
<td>( \varnothing )</td>
<td>Pile diameter</td>
<td>[m]</td>
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### Roman upper case letters

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<th>Letter</th>
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<tr>
<td>A</td>
<td>Area</td>
<td>$[m^2]$</td>
</tr>
<tr>
<td>B</td>
<td>Width of the abutment backwall</td>
<td>$[m]$</td>
</tr>
<tr>
<td>C</td>
<td>Detail fatigue category factor</td>
<td>$[N/m^2]$</td>
</tr>
<tr>
<td>E</td>
<td>Young's modulus, the modulus of elasticity</td>
<td>$[N/m^2]$</td>
</tr>
<tr>
<td>F</td>
<td>Lateral force</td>
<td>$[N]$</td>
</tr>
<tr>
<td>H</td>
<td>Abutment height</td>
<td>$[m]$</td>
</tr>
<tr>
<td>I</td>
<td>Moment of inertia</td>
<td>$[m^4]$</td>
</tr>
<tr>
<td>$I_{LT}$</td>
<td>Long term moment of inertia</td>
<td>$[m^4]$</td>
</tr>
<tr>
<td>$I_{ST}$</td>
<td>Short term moment of inertia</td>
<td>$[m^4]$</td>
</tr>
<tr>
<td>$K_a$</td>
<td>Active soil pressure coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>$K_p$</td>
<td>Passive soil pressure coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>L</td>
<td>Length</td>
<td>$[m]$</td>
</tr>
<tr>
<td>$L_b$</td>
<td>Bridge Length</td>
<td>$[m]$</td>
</tr>
<tr>
<td>$L_{equ}$</td>
<td>Equivalent cantilever length</td>
<td>$[m]$</td>
</tr>
<tr>
<td>M</td>
<td>Constant</td>
<td>[-]</td>
</tr>
<tr>
<td>M</td>
<td>Moment</td>
<td>$[Nm]$</td>
</tr>
<tr>
<td>N</td>
<td>Normal force</td>
<td>$[N]$</td>
</tr>
<tr>
<td>$N_f$</td>
<td>Number of cycles until failure</td>
<td>[-]</td>
</tr>
<tr>
<td>P</td>
<td>Axial force</td>
<td>$[N]$</td>
</tr>
<tr>
<td>R</td>
<td>Resulting normal force on abutment</td>
<td>$[N]$</td>
</tr>
<tr>
<td>T</td>
<td>Temperature</td>
<td>$[^{°}C]$</td>
</tr>
<tr>
<td>$T_0$</td>
<td>Datum temperature</td>
<td>$[^{°}C]$</td>
</tr>
<tr>
<td>$T_1$</td>
<td>Solar incremental temperature</td>
<td>$[^{°}C]$</td>
</tr>
<tr>
<td>$T_d$</td>
<td>Daily temperature</td>
<td>$[^{°}C]$</td>
</tr>
<tr>
<td>$T_E$</td>
<td>Non-linear temperature distribution</td>
<td>$[^{°}C]$</td>
</tr>
<tr>
<td>$T_K$</td>
<td>Characteristic temperature</td>
<td>$[^{°}C]$</td>
</tr>
<tr>
<td>$T_M$</td>
<td>Linearly varying temperature component</td>
<td>$[^{°}C]$</td>
</tr>
<tr>
<td>$T_N$</td>
<td>Effective bridge temperature (EBT)</td>
<td>$[^{°}C]$</td>
</tr>
<tr>
<td>$T_s$</td>
<td>Seasonal daily average temperature</td>
<td>$[^{°}C]$</td>
</tr>
<tr>
<td>W</td>
<td>Bending stiffness</td>
<td>$[m^3]$</td>
</tr>
<tr>
<td>W</td>
<td>Vehicle gross weight</td>
<td>[ton]</td>
</tr>
</tbody>
</table>
**Roman lower case letters**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>b</td>
<td>Fatigue strength exponent</td>
<td>[-]</td>
</tr>
<tr>
<td>c</td>
<td>Cohesion, or Fatigue ductility exponent</td>
<td>[-]</td>
</tr>
<tr>
<td>$c_{uk}$</td>
<td>Undrained shear strength in cohesive soils</td>
<td>[N/m$^2$]</td>
</tr>
<tr>
<td>d</td>
<td>Pile width</td>
<td>[m]</td>
</tr>
<tr>
<td>$k_{eh}$</td>
<td>Equivalent uniform lateral soil stiffness parameter</td>
<td>[N/m$^2$]</td>
</tr>
<tr>
<td>$k_h$</td>
<td>Lateral soil stiffness parameter</td>
<td>[N/m$^2$]</td>
</tr>
<tr>
<td>l</td>
<td>Critical pile length</td>
<td>[m]</td>
</tr>
<tr>
<td>$l_e$</td>
<td>Embedded length of $L_{equ}$</td>
<td>[m]</td>
</tr>
<tr>
<td>$l_u$</td>
<td>Not embedded pile length</td>
<td>[m]</td>
</tr>
<tr>
<td>m</td>
<td>Constant</td>
<td>[-]</td>
</tr>
<tr>
<td>n</td>
<td>Number of cycles, number of segments, number of piles</td>
<td>[-]</td>
</tr>
<tr>
<td>$n_h$</td>
<td>Coefficient of subgrade reaction</td>
<td>[N/m$^3$]</td>
</tr>
<tr>
<td>p</td>
<td>Soil pressure</td>
<td>[N/m$^2$]</td>
</tr>
<tr>
<td>$p_{act}$</td>
<td>Active soil pressure</td>
<td>[N/m$^2$]</td>
</tr>
<tr>
<td>$p_{pass}$</td>
<td>Passive soil pressure</td>
<td>[N/m$^2$]</td>
</tr>
<tr>
<td>q</td>
<td>Uniformly distributed load</td>
<td>[N/m]</td>
</tr>
<tr>
<td>$q_l$</td>
<td>Limit soil pressure</td>
<td>[N/m$^2$]</td>
</tr>
<tr>
<td>t</td>
<td>Pile material thickness</td>
<td>[m]</td>
</tr>
<tr>
<td>$t_d$</td>
<td>Time measured in days</td>
<td>[days]</td>
</tr>
<tr>
<td>$t_o$</td>
<td>Constant</td>
<td>[-]</td>
</tr>
<tr>
<td>z</td>
<td>Soil depth</td>
<td>[m]</td>
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Abbreviations

AADT  Annual Average Daily Traffic
AASHTO American Association of State Highway and Transportation Officials
BWIM  Bridge Weigh in Motion
CLT   Central Limit Theorem
DOT   Department of Transportation
EBT   Effective Bridge Temperature
FEM   Finite Element Method
FRP   Fibre Reinforced Polymers
LCC   Life Cycle Cost
MC    Monte Carlo
PDF   Probability Density Function
PMF   Probability Mass Function
SMHI  Swedish Meteorological and Hydrological Institute
WIM   Weigh in Motion
PART 1

Introduction and Literature Review
Low-cycle Fatigue of Steel Piles in Integral Abutment Bridges
1 Introduction

1.1 Background

There are several different factors that have to be taken into account when a bridge is designed. The most important factor is of course the safety aspect. The bridge must be designed to withstand all type of loads that is applied, traffic loads as well as wind and weather. Besides the safety aspects, the economical aspect is often the second most important. The cost of a bridge should be calculated as a life cycle cost, which takes into consideration all costs from construction costs to repair and maintenance costs. The costs of maintaining bridges are quite high and the society uses a lot of tax-money to keep the bridges in good conditions.

Conventional bridges are in general built with expansion joints and bearings. Expansion joints as well as bearings are weak points in a bridge structure. Leaking expansion joints are the most common reason for corrosion problems in bridges. These joints need to be maintained, repaired and also often replaced several times through the service lifetime of a bridge. If bridges were built without any expansion joints it would be possible to reduce the maintenance costs. This report is focused on bridges without any movable joints, integral abutment bridges.

Integral abutment bridges have other benefits besides lower maintenance costs. The foundations work can be simplified, implying lower construction costs. No expansion joints means no bump when a car enter or leave a bridge, the riding quality will be improved as well as the noise level for the travellers.

In the USA they have been constructing integral bridges during more than half a century. The maximum length of integral bridges has been increasing, and some states allows composite bridges up to a length of almost 200 m. Integral bridges with concrete superstructures are allowed to be even longer, a 358 m long integral bridge with a concrete superstructure has been constructed in Tennessee.

Today’s problem, seen from a Swedish perspective, is that a conventional elastic analysis fails to explain how integral abutment bridges work. The Swedish National Road Administration is far more conservative, than the USA, concerning integral abutment bridges. This leads to a long and expensive design process where the National Road Administration determines the limits on a case to case basis. A development of general codes, rules or guidelines for integral bridges would simplify the design process.

Low-cycle fatigue has not been experienced as a problem in the USA. However, piles in integral abutment bridges are sometimes designed to experience stresses above the pile yield strength, and plastic hinges are allowed to develop. If the cyclic loads that act on the piles, for instance due to lateral temperature movements, are giving reversible strains exceeding the yield strain. Then low-cycle fatigue could be a possible failure mode.
1.2 Aims and Scope

The first aim of this report is to perform a literature review of integral bridges in general and especially low-cycle fatigue of piles in integral abutment bridges. The literature study is focused on the following areas: integral abutment piles, temperature movements, low-cycle fatigue, traffic loads, and the Monte Carlo method. These areas are chosen since certain knowledge in these fields is believed to be necessary in order to perform the second aim.

The second and primary aim is to investigate if, how and when low-cycle fatigue failure will be a possible failure mode for piles carrying integral abutment bridges. The varying pile strains, as a result of thermal movements, traffic loads, etc, are simulated by a Monte Carlo simulation, in order to achieve a strain-time sequence during the bridge service lifetime. The strain sequence is evaluated and the fatigue life is predicted.

Some delimitations of the problem are established. The study is limited to integral abutment bridges with stub abutments, deflections of the abutments are not considered. Thermal movements are studied in a Swedish climate. One example bridge is studied, and several parameters are varied in order to investigate their influences. The studied bridge is a composite bridge, no concrete bridges are studied. The soil-pile interaction model is chosen to make it possible to perform calculations without any external computer program. The analysed pile cross-sections are dimensioned and chosen by the bridge designer, and the only failure mode studied in this report is fatigue failure.

1.3 Structure of the Thesis

The first part of this report contains a literature review of integral bridges in general, and especially the thermal behaviour of the bridge structure and the low-cycle fatigue of the abutment piles. The Monte Carlo simulation method is also studied. The second part contains calculations of cumulative pile fatigue in an integral abutment bridge over the Leduån stream. A Monte Carlo simulation of the pile fatigue is presented.

Part 1

Chapter 2 - Integral Bridges
The concept of integral abutment bridges is defined, and typical integral bridges are described. Benefits and drawbacks of integral bridges are summarised.

Chapter 3 - International Experiences of Integral Bridges
Different countries views of integral bridges and their design, advantages, limitations, etc. are presented and a couple of bridges are shown as examples.

Chapter 4 - Piles
Piles supporting integral abutment bridges are described in this chapter. Different pile materials and cross-sections are described and compared. The importance of pile orientations is also discussed, and some simplified pile design models are presented.
Chapter 5 - Bridge Temperature
The effect of varying bridge temperatures in integral abutment bridges is discussed and factors that influencing bridge temperatures are described. The concept of Effective Bridge Temperature (EBT) is presented and temperature movements’ effects on the piles are described. How bridge design codes deals with thermal actions is also reviewed, and a simplified structural analysis of thermal movements is presented.

Chapter 6 - Low-cycle Fatigue
Low-cycle fatigue is described and some methods of predicting the fatigue lifetime are briefly discussed. The concept of cumulative fatigue damage is also presented.

Chapter 7 - Traffic Loads
Traffic fatigue loads given in bridge design codes are presented. Measurements of real loads are discussed as an alternative to the values given in codes. The traffic load measurement techniques WIM and BWIM are briefly described and discussed.

Chapter 8 - Monte Carlo Simulation
This chapter is a review of the Monte Carlo simulation method. The purpose and benefits of a simulation is presented and the theories behind the method are summarised.

Part 2

Chapter 9 - Example Bridge - Leduån Bridge
Presentation of the Leduån Bridge, which has been used in the calculations throughout the report. Type of bridge, materials, dimensions, geometry, etc. is presented.

Chapter 10 - Temperature Models
If a pile strain simulation shall be performed, daily and seasonal temperature variations have to be modelled in one way or another. In this chapter a couple of proposals to temperature models are given. Their benefits and drawbacks are discussed and one model is chosen for the temperature simulation.

Chapter 11 - Temperature Simulations
The Monte Carlo simulation models for daily temperature variations and vertical temperature differences are presented. The simulated temperatures are compared to real measurement and differences are discussed.

Chapter 12 - Soil Calculations
Soil calculations are performed according to a simplified soil model proposed by Abendroth and Greimann. The equivalent uniform lateral soil stiffness parameter is calculated for two different soil models. This parameter is later used to calculate the equivalent cantilever length.

Chapter 13 - Loads and Rotations
This chapter presents the loads that are assumed to act on the bridge. Permanent loads are calculated as well as variable loads. Forces, rotations and movements which occur as a result of the loads are calculated and summarised.
Chapter 14 - Maximum Strain Calculations
The maximum pile strains are calculated in this chapter, in order to find out how large strain variations that can be expected in the simulation of pile strains. The results of these calculations and the probability of a low-cycle fatigue failure are discussed.

Chapter 15 - Fatigue Simulation Model
The fatigue simulation model is presented and the traffic load model that is used in the simulation is described. The fatigue life calculation model is discussed, and different types of strain cycles are defined. Cycle-counting techniques are also discussed, and the Rain-flow method is presented.

Chapter 16 - Fatigue Simulation Results
The results from the strain simulations and the fatigue calculations are presented.

Chapter 17 - Conclusion and Discussion
This chapter concludes this report. The conclusion from the fatigue calculations are presented and discussed. Possible sources of errors are discussed and some further research proposals are given.
2 Integral Bridges

Conventional bridges are in general designed with expansion joints and bearings. These components are weak points in a bridge structure, and associated with large maintenance costs. A bridge without these weak points would be preferable. This thesis is focused on bridges without movable joints, integral abutment bridges.

2.1 Definition of Integral Bridges

Integral bridges are bridges without expansion joints between the abutment and the superstructure. Integral bridges have instead their continuous decks and girders integrated into the abutments. Rigid joints between the different structural members of an integral bridge can be constructed as shown in Figure 2:1.

1. Vertical piles are driven into the ground, generally in a straight single row.
2. The lower part of the abutment wall is cast. The upper part of the piles can either be surrounded by a pilecap or only by the abutment wall.
3. The bridge girders are mounted and anchored at the top of the lower part of the abutment wall.
4. The concrete deck slab and the top of the abutment walls are cast. The walls are cast in the same stage as the end stage of the bridge slab.
5. The end parts of the bridge girders will then be surrounded by concrete. This gives a rigid frame structure, which will act as one structural unit.

Figure 2:1 Illustration of how an integral abutment can be constructed.

There are a couple of different names of this type of bridge in the literature. Integral abutment bridges, jointless bridges, continuous bridges and rigid frame bridges are sometimes used instead of integral bridges.

This type of bridge can be constructed as single or multiple span bridges. Each abutment is normally supported by a single row of vertical flexible piles, which creates an interaction between the abutments and the soil. In general, there are no longitudinal battered piles in an integral abutment bridge. The horizontal flexibility in the vertical piles makes it possible for the abutments to move when the length of the bridge changes as a result of variations in temperature.

There is no world wide definition of the term integral bridge. In the USA it usually refers to bridges with stub abutments which are connected to the superstructure without any joints. The abutments are generally supported by a single row of flexible piles. The Highways Agency in the UK has a similar definition of the term, but they are not only using it for bridges with piled stub-abutments (UK Highways Agency 2003). Figure 2:2 is an illustration from UK Highways Agency’s design manual for integral bridges. It shows which types of integral constructions they have made their manual for.
A bridge shall support all dead and live loading that are relevant in the specific case. If an integral bridge is designed, instead of a non-integral, additional strains and stresses are created in the bridge elements. These are a result of thermal movements of the bridge, and creep and shrinkage of the concrete. The length of the bridge will be increasing and decreasing when the temperature is changing. When the length of the bridge is changing, forces and movements will be induced in the abutments and the piles, which will deflect and rotate. The piles, which are supporting the abutments, will be subjected to shear forces as well as bending moments (Arsoy 1999). It is necessary to estimate the forces that are induced in the abutment and the piles. The movements of the abutments will cause an increasing or decreasing soil pressure in the backfill, the interaction between soil and abutment walls must also be considered. (Abendroth and Greimann 2005)

Variations in bridge temperature appear both daily and seasonally and will lead to cyclic loading of the piles. The piles will deflect under the loading and there might be plastic deformations. The piles could then be subjected to low-cycle fatigue, and this may result in a reduction of the bridge service lifetime. (Dicleli and Albhaisi 2004)

The trend in the USA has been towards longer and longer integral bridges. In view of the fact that thermal movement is a function of the bridge length, there will be larger and larger movements and forces as well. During the design phase, it is necessary to be able to estimate the forces that are acting on the piles in a proper way. To be able to make a good estimation, there is a need of information about the thermal behaviour of the abutment-backfill system and the soil-pile system. One of the factors that are determining the maximum length of integral bridges is the ability of the piles to take lateral movements. It is necessary to keep the piles stresses low, in order to build longer integral bridges. (Arsoy 2000, Dicleli and Albhaisi 2004)
2.2 Types of Integral Bridges

The simplest form of an integral bridge is a portal frame structure, but there are also several quite complicated forms of integral bridges as well. Several different types of abutments and other technical solutions have been used in integral bridges through the history. This report is focused on integral bridges with “fully” integral abutments. Figure 2:3 shows a fully integral abutment to the left, and a semi integral abutment to the right.

![Diagram of fully and semi integral abutments]

A fully integral abutment transfers all movements, moments and forces through a rigid joint between the superstructure and the abutment. A semi integral abutment is only restraining translations but not rotations between the superstructure and the abutments.

2.3 Why Integral Bridges?

When a bridge is designed it is important to calculate the life cycle cost (LCC), in order to find out which alternative that is most economical in the long term. Expansion joints and expansion bearings are two factors that are influencing the LCC. There are a lot of costs connected to these two components. They are expensive to buy and install in the first time, and then they need to be maintained, repaired and often also replaced. Leaking expansion joints and seals are in fact the most common reason for corrosion problems in bridges (Mistry 2005). Water from the road, containing dirt and salt, can leak through the joints and then attack the bridge girders, bearings and the reinforced concrete. The expansion joints are also heavily loaded by axle loads from vehicles that cross the bridge, and snowplow damages are not rare.

Many of the most expensive maintenance activities are connected to problems with the expansion joints. Therefore, it would be preferable if a bridge could be built without any movable joint. Henry Derthick, former Engineer of Structures at Tennessee Department of Transportation, declared in the sixties that “The only good joint is no joint” (Burdette et al. 2005). This philosophy has later been adopted by other states and countries as well.
2.3.1 Advantages

Construction costs: It is often more economical to construct integral abutment bridges instead of bridges with joints and bearings. The construction time can often be reduced, since fewer piles are needed and no battered piles, and the time consuming installation of expansion joints and bearings are eliminated.

Maintenance costs: Leaking expansion joints is one of the most common reasons to corrosion problems. Expansion joints and bearings need to be maintained, repaired and replaced. Integral bridges have no expansion joints or bearings and are therefore less expensive to maintain.

Modification costs: It is easier and cheaper to modify an integral bridge, for instance widening.

Riding quality: No expansion joints, means no bump when a vehicle enter or leave a bridge. This gives a smoother ride for the passengers and the noise level is reduced.

Earthquake resistance: The most common cause of damage to a bridge due to seismic events is loss of girder support. That problem is eliminated in an integral bridge construction.

2.3.2 Disadvantages

High pile stresses: High stresses can occur in the abutment piles and plastic hinges can be developed. If plastic rotations occur there can be problems with low-cycle fatigue.

Limitations: There is a limited range of applications for integral abutment bridges. An increasing length gives increasing movements of the abutments. This relation causes a maximum length for integral bridges. The skew angle of the bridge abutment is also limited.

Lack of knowledge: There is a lack of knowledge about several parts of the bridge construction. The knowledge about the soil-abutment interaction and the pile-soil interaction are two areas that need to be investigated.

No general codes: There are no general codes, rules or guidelines available in many countries. In the USA they have been constructing integral bridges during more than half a century, but they still do not have any general guidelines.
3 International Experiences of Integral Bridges

3.1 USA

A study made in the USA by Greimann et al. (1984), shows that 29 of 52 states design agencies permits the design and construction of integral bridges. But there were only two states that calculated the pile stresses due to the lateral thermal movements. Most of the remaining states neglected these stresses, while a few states demanded some construction details that should reduce these stresses. One example of how pile stresses could be reduced is to drive the piles into pre-drilled oversized holes, which are backfilled with loose sand. (Abendroth and Greimann 2005)

Since the study made in the early eighties, several states have adopt integral abutment bridges as an alternative when the conditions allow such a structure. Mistry (2005) establishes that there are now at least 40 states that are constructing some form of jointless bridges. The trend seems to be moving towards integral abutment bridges, but most of the bridges are still constructed with expansion joints. In 2004, The Federal Highway Administration conducted a survey of integral abutment and jointless bridges. This survey indicates that the number of integral bridges have increased by more than 200% during the last ten years (Maruri and Petro 2005). Many states have nowadays declared that integral bridges shall be considered as the first choice when a new bridge is designed or an old bridge is replaced (Conboy and Stoothoff 2005, Yannotti et al. 2005).

Integral bridges have been constructed successfully for decades, although there are no common rules of how the design and construction shall be performed. The design and analysis of integral abutment bridges have mostly relied on previous experiences from the same type of structure. (Maruri and Petro 2005)

Dicleli and Albhaisi (2004) have summarised the maximum length of integral bridges that different departments of transportations allows, see Table 3:1. A longer list of maximum bridge lengths is presented in Appendix N. There are some exceptions to the maximum length limits. For instance, a 358 m long integral bridge made of concrete has been constructed in Tennessee in 1997, although the upper limit is set to 244 m. These upper limits shall not be treated as the “truth” of how long an integral bridge can be. The limits are often based on experiences from previous constructed bridges. Dicleli and Albhaisi (2004) have, like other researchers, developed methods for calculating the maximum length of integral bridges. Their method is mainly dependent on which type of piles that are used and how large temperature variations that are expected.

<table>
<thead>
<tr>
<th>Department of Transportation</th>
<th>Maximum length</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Composite bridges [m]</td>
</tr>
<tr>
<td>Colorado</td>
<td>195</td>
</tr>
<tr>
<td>Illinois</td>
<td>95</td>
</tr>
<tr>
<td>New Jersey</td>
<td>140</td>
</tr>
<tr>
<td>Ontario, Canada</td>
<td>100</td>
</tr>
<tr>
<td>Tennessee</td>
<td>152</td>
</tr>
<tr>
<td>Washington</td>
<td>91</td>
</tr>
</tbody>
</table>

Table 3:1 Maximum length limits for integral abutment bridges (Dicleli and Albhaisi 2004).
3.2 UK

The UK Highways Agency has some recommendations of when an integral bridge shall be considered. The following sentences are taken from their Design Manual for Roads and Bridges, volume 1 section 3 part 7.

“Continuous structures have proved to be more durable than structures with simply supported decks.”

“In principle, bridges with lengths not exceeding 60 m and skews not exceeding 30° shall in addition be designed as integral bridges, with abutments connected directly to the bridge deck without movement joints for expansion or contraction of the deck.”

The design manual is also addressing that these sentences are valid for bridges of steel, concrete and composite structures. With the requirement that the cyclic movements, that are induced by temperature variations, do not exceed ± 20 mm for each abutment. Piles in integral abutments are not allowed to be inclined if the abutments move laterally, and piles shall be designed to withstand both axial and lateral forces. (UK Highways Agency 2003)

3.3 Australia

Integral bridges are not so common in Australia, and there are no national guidelines for the design of them. It is up to the Australian engineers to use their engineering knowledge in the best way. This is generally done by using the experiences that other countries have had. USA and UK are the two countries whose methods often are used as references. The Gilles Street Bridge in an example of an integral bridge in Australia, see Figure 2:1. It was constructed in 1995, and the 59 m long superstructure is made of six precast concrete T-girders and a concrete deck slab, which was cast in place. (Connal 2004)

One opinion in Australia is that integral bridges give most benefits in areas with climates that involve ice and snow, because of the bad combination of leaking joints and salt used as de-icing agent. Snow and ice on Australian roads are a very rare phenomenon, and de-icing agents are not used. Therefore, the Australian Highway Agency does not think that they will have the same benefits as UK and USA, where the climates are quite different with more varying weather conditions. This is one reason to why Australia does not construct integral bridges so often and does not have any special guidelines for integral bridges.

Figure 3:1 Gilles Street bridge in Ballarat, Australia (Connal 2004).
3.4 Japan

In an article written by Japanese researchers in 2002, integral abutment bridges are mentioned as a new technology of constructing composite bridges. An example of an integral bridge constructed in Japan is shown in Figure 3:2.

The Nishihama Bridge has a composite superstructure consisting of eight steel girders and a concrete deck slab. The abutments are made of concrete and are supported by five steel piles with the rather large diameter of 800 mm. An interesting thing with this bridge is that it is located in the area of a steel mill, and vehicles with a weight of 90 tonnes are crossing the bridge frequently. The high traffic loads could be a problem in the combination with large temperature movement. The Nishihama Bridge is however quite short and located in an area where the temperature differences over a year are not that big. A difference of 30°C has been measured in the steel girders between January and August. Figure 3:3 shows a side view drawing of the bridge and a view of one of the abutments. (Nakamura et al. 2002)

Figure 3:2 Nishihama Bridge (Nakamura et al. 2002).

Figure 3:3 The Nishihama Bridge, side view drawing to the left and abutment view drawing to the right.
3.5 Sweden

Integral abutment bridges are not that common in Sweden. A couple of integral bridges have been constructed in the last few years, and they seem to become a more popular alternative. The lengths of the bridges have so far been kept rather low. The Swedish National Road Administration is rather conservative, concerning integral abutment bridges, compared to some states Department of Transportation in the USA. In some states they design bridges with the line of argument “it simply works”. Integral bridges are very difficult to analyse, and this is probably the main reason of why they have not yet become common in Sweden. Conventional elastic analysis fails to explain how the bridges work. This leads to a long and expensive design process where the National Road Administration determines the limits on a case to case basis. A development of general codes, rules or guidelines for integral bridges would simplify the design process. (Pétursson and Collin 2002)

Hans Pétursson has in a post-graduate project developed some guidelines of how integral abutment bridges can be designed (Pétursson 2000). These guidelines have been used in the design of a couple of integral bridges in Sweden. Some of them are presented below.

Bridge over Fjällån

- The bridge is a single span composite bridge with a span length of 37.15 m and a width of 9.0 m. Two welded steel girders, c/c 5.0 m, form the superstructure together with the concrete deck slab.
- 8 X-piles (X180x24) were used in each abutment. These piles were oriented 45° to the bridge longitudinal axis, in order to minimise the bending stresses.
- Constructed in year 2000.
Bridge over Hökviksån

- The bridge is a single span steel arch bridge with a span length of 42 m, and a width of 7.0 m. An old conventional concrete arch bridge, from 1922, has been replaced by the new integral bridge.
- 8 steel X-piles with a width of 200 mm and a thickness of 30 mm were used in each abutment. The outermost piles are inclined 4:1 to take care of transverse loading. All of the piles were oriented 45° to the bridge longitudinal axis.
- Constructed in year 2004.

Bridge over Leduån

- This bridge is used in the calculations throughout this report, and it is described in detail in Chapter 9.
- The bridge is a single span composite bridge with a span length of 40 m and a width of 5.0 m. 6 steel pipe piles, RR170x10, were used in each abutment.
- Constructed in year 2006.
Low-cycle Fatigue of Steel Piles in Integral Abutment Bridges
4 Piles

Integral abutment piles are in general designed to have sufficient vertical capacity and low flexural stiffness. The stiffness of the piles should be low in order to minimise the flexural effects due to lateral movements and rotations of the abutments. The lateral movements are mainly caused by variations in the temperature of the superstructure, but other internal and external loads are also giving contributions to the movements. Lateral movements together with the rigid connections between piles and abutment, gives maximum pile moments in the connection between pile and abutment where the fixity is highest. Both theory and field studies (Abendroth and Greimann 2005) have showed that piles in integral abutment bridges will sometimes experience strains above their yield strain. The plastic strains are mainly a result of the thermal movements of the abutments. It is therefore necessary to predict the rotation capacity of the piles and their capacity to withstand repeated plastic deformations, low-cycle fatigue.

The stiffness of the foundation soil is a very important parameter for the cyclic displacement capacity of steel piles in integral bridges. A stiffer soil can results in a huge loss of displacement capacity. Therefore, increasing soil stiffness gives a decreasing maximum length for integral bridges. The soil stiffness must be reduced in one way or another if the maximum length of integral bridges shall be increased (Dicleli and Alhaisi 2004). Oversized pre-drilled holes are one way of reducing the horizontal resistance against lateral displacements at the top of the piles. These holes are backfilled with low stiffness material which surrounds the top of the piles. Examples of loose materials that are used is sand and bentonite slurry (Dicleli and Alhaisi 2003), compressible foam has also been used (Connal 2004). A study in the USA from the early 1980’s showed that only four of twenty-nine states, that were building integral bridges, were normally using pre-drilled holes. A newer survey from 2000, which Abendroth and Greimann (2005) refer to, shows that twelve of thirty states now are demanding a use of pre-drilled holes.

4.1 Pile Materials and Cross-sections

Several different pile materials and cross sections have been used in integral bridges. Steel is the most common used pile material in integral bridges and the most common cross-section seems to be H-piles. Concrete is also used as a pile material but has some limitations. Composite fibre materials are not used at the moment, but might be a competitive alternative in the near future. Figure 4:1 illustrates the cross-sections that are described in the following sections.

4.1.1 Steel Piles

Steel piles can take cyclic stresses at least up to their yield stress capacity, provided that the used cross-section will not experience local buckling. If the piles have sufficient rotation capacity and plastic hinges are allowed, then it would be possible to tolerate strains which exceed the yield strain. Plastic strains can however lead to low-cycle fatigue failure and this should be taken into consideration when the piles are designed. The influence of the corrosion on steel piles must also be taken into account, since the unaffected cross section area will be time dependent.
H-piles
H-piles seem to be the first choice for integral bridges in the USA, especially in longer bridges (Bakeer et al. 2005). Arsoy (2000) made cyclic load tests of steel H-piles, pipe piles and prestressed reinforced concrete piles. His test results show that steel H-piles is the best choice for integral bridges, among the tested piles. This fact seems to be well known since most of the countries and states that are building integral bridges prefer steel H-piles. The opinion of how the H-piles shall be orientated, in weak or strong axis bending, is varying.

Pipe piles
Steel pipe piles are an alternative to steel H-piles. Cyclic load tests on H-piles and pipe piles with the same width have been performed by Arsoy (2000). These tests show that pipe piles probably would survive the cyclic loading, and the abutment seems to be the first part to fail if there would be a failure. The tested piles had the same pile width, and that lead to a rather stiff pipe pile compared to the H-piles oriented in weak axis bending. The pipe pile had an area which was 71% larger than the H-pile, and the moment of inertia was almost 7 times higher. One conclusion drawn by Arsoy is that stiff piles, like pipe piles, shall not be recommended for integral abutment bridges. Cross-sections with lower flexural stiffness are preferable, like H-piles oriented in weak axis bending.

X-piles
Cross-shaped steel piles, X-piles, have been used in integral bridges in Sweden. The X-shaped piles are driven in a straight line and rotated $45^\circ$ in order to minimize the bending stresses, see Figure 4:1. (Pétursson and Collin 2002)

4.1.2 Reinforced Concrete Piles
Concrete piles are often used in rather short bridges, at least in the USA. The large disadvantages of concrete piles are the low tensile strength and the possibility of suddenly failures. As long as the yield stress is not exceeded, then concrete piles fulfill their purpose. But, a suddenly failure can occur if the stress just slightly exceeds the yield strain. Cyclic lateral displacements of the top of the piles lead to a development of cracks in the concrete due to the tensile stresses. These cracks will decrease the vertical load carrying capacity of the piles, which will be dependent on the number of cycles (Arsoy 2000). Since the mentioned disadvantages are well known, many states have established upper length limits for integral bridges with concrete piles. For instance, Illinois allows concrete piles up to a bridge length of 61 m (200 feet). (Bakeer et al. 2005)

4.1.3 Steel Pipes Filled with Concrete
Cast-In-Place piles (CIP) are driven steel pipes which later are filled with concrete and some reinforcement in the upper part of the piles. These piles are like the concrete piles generally used only in short bridges. Minnesota Department of Transportation allows these piles in integral bridges with a bridge length less than 45 m. (Huang et al. 2004)
4.1.4 Fibre Composite Piles

The cross-section area for steel and concrete piles will be decreasing as time goes, due to corrosion and deterioration. Piles made of composite materials can be designed to resist corrosive soil and other hostile environments. Composite materials might therefore be a competitive alternative to the traditional pile materials. Advantages of piles made of fibre-reinforced-polymers (FRP) are corrosion and chemical resistance, high oriented-strength structural shapes, durability, and low maintenance. FRP is a very expensive material compared to concrete, and a combination of these materials can be used to get more economical FRP piles. The FRP piles can be made as hollow cross-sections that are filled with concrete, which increases the strength and ductility. A study made by Jaradat (2005) resulted in the conclusions that rectangular piles gives a possibility of geometrical optimization unlike circular piles. The cross-sections can be optimized for lowest stress and stiffness. The rectangular shaped FRP piles behaved better than circular piles when they were experiencing lateral displacements. The circular piles experienced local buckling whereas the rectangular piles performed without local buckling. This is probably an effect of the two webs which provides lateral support against local buckling. The distance between webs and flanges can be designed to optimize a pile to a certain bridge and soil condition. (Jaradat 2005)

FRPs might be a common pile material in the future, but the present use of FRP piles seems to be very limited. US Navy has the largest usage of FRP piles in the USA today. They use them mostly as marine fenders, because of there high strength and durability.

![Figure 4:1 Illustration of different pile cross-sections.](image)

4.2 Pile Orientation

There have been different opinions about how single symmetric integral abutment piles, like H-piles, shall be oriented. In the early 1980’s more than half of states in the USA, which allowed integral bridges, oriented their piles for strong-axis bending due to the thermal movements. (Abendroth and Greimann 2005)

In a survey ordered by The Federal Highway Administration in the USA, in year 2004, the Departments of Transportations were asked how they oriented the piles in their integral bridges. The result from this survey is quite different compared to the study from the early
1980’s, which Abendroth and Greimann (2005) refer to. Nowadays, most of the states are orienting integral abutment piles for weak axis bending, see Figure 4:2. But it is obvious that there are no uniform rules that are applied all over the USA, each state makes their own decisions. Australia follows the main trend in the USA and orientates their piles for weak axis bending (Connal 2004). Figure 4:3 shows how the orientation of piles was varying from state to state in 2004. (Maruri and Petro 2005).

The reason to orient the pile for weak axis bending is mainly to minimize the stresses in the abutments. For a given displacement of the abutment, a pile oriented for strong axis bending will induce higher stresses in the abutment than a pile oriented for weak axis bending. It is also done in order to make sure that local buckling of the flanges shall not occur, even if the soil is not supporting the pile laterally. (Arsoy 2000, Huckabee 2005)

Dicleli and Albhaisi (2003) have studied the effect of cyclic thermal loading on the performance of steel H-piles in integral bridges with stub-abutments. One of their conclusions is that the orientation of the piles has only a negligible effect on the displacement capacity of the integral bridge. Their study was made on bridges with stub-abutments, and it is possible that this conclusion is not valid for bridges with larger abutment heights.
4.3 **Pile-Abutment-Girder Interaction**

The pile-abutment-girder interaction is very important in the design of the piles. Rigid connections will transfer all forces and movements down into the piles. Hinged connections can be used in order to transfer only vertical and shear forces to the piles, and no moments.

4.3.1 **Rigid Joints**

One way of constructing a rigid connection between piles and girders is to cover the driven piles with a pilecap, or a lower part of the abutment backwall. The girders can then be mounted on top of the pile cap, and fixed to the abutments on levelling bolts that have been anchored in the pile cap, see *Figure 4:4a*. The ends of the girders are later surrounded by concrete, when the top of the abutment backwall is cast. But there are different techniques in the creation of a pile-abutment connection. When steel girders are used, some states in the USA prefer a welded connection between piles and girders, see *Figure 4:4b*. This technique is nowadays used in for example Maine, and has previously been used in many other states. It was a common way of constructing integral abutments in New York a decade ago, but they experienced some problems with this technique and prefers other types of connections nowadays. When the piles are driven they have to be very close to their planned position, if the girders shall be welded on top of them. This means that piles often must be driven within a tolerance of 2-3 cm, and this can be hard to achieve in difficult pile driving condition. (Conboy and Stoothoff 2005, Yannotti et al. 2005)

![Illustrations of two different techniques of designing the pile-abutment-girder connection for steel bridges with integral abutments (Yannotti et al. 2005).](image)

It has been proved that constructions without welds between piles and girders are easier to construct, and no differences in performance have been detected (Conboy and Stoothoff 2005). Due to these facts, it is hard to understand why some states are still using welded connections.
4.3.2 Hinged Joints

Another approach is to use a hinge between abutment and piles. The hinge transfers only vertical and shear forces to the piles, no bending moment is transferred. An example of a bridge with abutments constructed with this technique is Gillies Street Bridge, Australia. Figure 4:5 shows one of the abutments from that bridge and the hinged connection between the abutment and the concrete piles that was used. (Connal 2004)

![Figure 4:5 Abutment with hinged-piles (Connal 2004).](image)

The pin connection used in Gilles Street Bridge is illustrated more in detail in Figure 4:6. The pin connection was made of galvanized dowel bars, which was anchored in both the concrete pile and the pile cap. Polystyrene sheets were used as joint filling in order to avoid crushing of the concrete when the pile cap is rotating due to the applied moments. To make sure that the lateral forces are not getting to high in the top of the concrete piles, the upper 2 m were wrapped with 50 mm thick compressible foam.

![Figure 4:6 Illustration of the pin connection used in Gilles Street Bridge.](image)

Virginia Department of Transportation (VDOT) has been working a lot with jointless bridges and has very good experiences from this type of bridges. VDOT prefers steel H-piles oriented in weak axis bending, and the abutments shall be designed in a way that reduces the pile stresses. One way of reducing the pile stresses is to construct a moment relief hinge in the abutment wall. VDOT has been developing a moment relief hinge based on a shear key along the joint. This type of hinge has later been modified after some tests made by Arsoy (2000). Figure 4:7 illustrates both the original and the modified hinge. (Weakley 2005)
The modified hinge is more flexible to rotations and consists of strips of neoprene along both sides of the line of dowels. The rest of the joint is filled with some joint filler, for example sponge rubber. The vertical forces will be transferred from the upper part of the abutment through the neoprene and down into the pilecap, and the dowels will transfer the shear forces.

Arsoy (2000) performed full scale laboratory tests of the original hinge construction and the modified hinge. The hinges were tested both with static and cyclic lateral loading. The shear key in the original hinge construction failed already at the static test. Analysis of the data showed that the hinge did not work as a hinge. The abutment and the pile cap rotated as one singular unit until the shear key failed. The bond between the upper and lower part was almost as strong as if they would have been cast together. The modified hinge did not show such behaviour in the tests, it behaved more as a hinge. The cyclic load test showed no sign of fatigue failure after more than 27000 cycles, which should simulate the thermal movements during 75 years. The bending stress during these cycles were a bit higher than the yield strength of the dowel bars. The original hinge with the failed shear key did not show any further damage after the cyclic test. A failure of the shear key is therefore not expected to result in a collapse of the bridge.

The rotational stiffness of the hinged abutments seems to be dependent on the abutments rotation angle. When the rotation gets larger, then the rotational stiffness seems to go towards a low constant. Arsoy (2000) draw the conclusion that hinged abutments actually are reducing the pile moments quite a lot. This technique can therefore be useful in order to construct longer bridges with integral abutments, without getting to high bending moments in the piles.
4.4 Simplified Pile Design

With the computer programs that are available today it would be possible to analyse the soil-pile-abutment-girder interaction in a realistic way. A non-linear Finite Element Method (FEM) could be used to calculate the pile stresses by taking into consideration the real material properties of the steel and soil (Pétursson 2000). This type of calculation is generally time consuming and is not an alternative in this report when a Monte Carlo (MC) simulation of the loads and movements shall be performed. The possibility of calling upon results from another computer program, like any FEM-program, is limited. It might be possible but would require some computer programming or another tool for the MC simulation. A simplified model of the pile stresses/strains is necessary in order to be able to perform a MC simulation of the pile strains. This section is therefore focused on simplified methods that are used to calculate pile stresses/strains.

According to Huckabee (2005), pile design for integral abutments involves two main criteria that must be fulfilled, the geotechnical and the structural criteria. The geotechnical design of the piles are mainly focused on the length of the pile embedment that are necessary, and the end bearing capacity in order to transfer vertical loads into the ground. The structural design criteria is focusing on the capability of the piles to carry vertical loads without experiencing local or global buckling when they are bended due to the abutment movements and rotations.

Generally, buckling is not critical in the design of embedded piles. The soil has to be very soft if the lateral restraint shall be insufficient to prevent buckling. Problems with buckling in working piles are very rare, and are often more likely to happen during the handling and pitching of the piles, than during the pile service lifetime. Sections with low rotation capacity shall not be chosen to avoid buckling problems during the installation. (Fleming et al. 1992)

The piles are not just oriented differently in the states in USA. They are also designed in different ways. A survey in the USA shows that the capacity of the piles is calculated with different forces and methods. The vertical forces alone are used by 41% of the states to determine the capacity of the piles. There will not be only vertical forces in the reality, the expansion and contraction of the superstructure will also cause lateral forces and bending moments. Therefore, it is more common to use a combination of axial and bending capacity, 51 % of the states are using this combination in their design calculations. (Maruri and Petro 2004)

4.4.1 Massachusetts’ Way of Designing Piles

Massachusetts is one of the states that require that the pile capacity must meet both the axial load criteria and the bending criteria stated by American Association of State Highway and Transportation Officials (AASHTO). The standard formulas that AASHTO suggests have been modified in order to deal with the extra rotational capacity of a compact HP-pile. The method developed in Massachusetts is based on the Abendorf and Greimann approach which is presented in the next section. Massachusetts has introduced a “Coefficient of Inelastic Rotation Capacity”, \( \theta \), as a step towards a design model that describes the real situation in a better way. Compact HP-piles sections have an extra rotational capacity that is three times the

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1 The Monte Carlo method is described more in detail in chapter 8 Monte Carlo Simulation.
elastic rotation capacity when they are bent in the weak-axis direction. The piles in an integral abutment bridge can theoretically be subjected to movements, which cause plastic rotations in the top of the piles that are three times the elastic rotation, without buckling or increased stresses in the compression flange. Even if the piles theoretically can take rotations that are 3 times bigger than the elastic rotations, they do not design piles for those large movements. Instead, a coefficient with a value of 1.75 is used. This gives a conservative formula, with a rather large safety margin. The extra coefficient, $\theta_i$, is only used in weak axis bending. If the bridge is skewed and both axes are in bending, then the strong axis term will remain unmodified. The formula for the design of the piles is expressed as follows: (Huckabee 2005, Conboy and Stoothoff 2005)

$$\frac{P_u}{0.85 \cdot A_s \cdot F_y} + \frac{M_y}{\theta_i \cdot M_{wy}} + \frac{M_x}{M_{wx}} \leq 1.0.$$  (4.1)

$P_u$ = Applied axial load determined from analysis.
$A_s$ = Pile cross-section area.
y = weak axis bending, x = strong axis bending
$M_y, M_x$ = Applied moment determined from analysis and P-Δ moment.
$M_{wy}, M_{wx}$ = Maximum moment strength based on slenderness criteria.
$\theta_i$ = Coefficient of inelastic rotation capacity. 1.75 for compact sections. 1.0 for non-compact-sections.

4.4.2 The Abendroth – Greimann Approach

Abendroth and Greimann (1989) suggest a simplified design approach for the design of integral abutment piles. This approach is based on analytical and experimental investigations, and has been verified by Girton et al. (1991) through field monitoring of two integral abutment bridges in Iowa. Arsoy (2000) notes that the model appears to be overly conservative since it does not consider the effects of the abutment-backfill interactions.

The method gives two alternatives of how to design integral abutment piles. Calculations of bending moments differ between the alternatives, while the calculations of axial forces are the same. The total axial force that is acting on a pile is the sum of the vertical gravity load, $N_w$, and the vertical load component due to the thermal expansion, $N_T$.

$$N = N_w + N_T$$  (4.2)

**Alternative 1**

This alternative is based on elastic behaviour only, and do not take into consideration the reserve strength due to plastic hinges. This alternative can be useful in the design of concrete and timber piles, and steel piles with cross-sections that have moment-rotation capacities that are too low. If steel piles have a significant moment-rotation capacity and they still are designed by **Alternative 1**, then they will be designed very conservatively.

The bending moment applied at the top of the pile will be the sum of the moment from the gravity loads and the moment due to thermal extraction and contraction of the superstructure.

$$M = M_w + M_T$$  (4.3)
The gravity induced moment, $M_w$, can be calculated conservatively by doing some simplifications of the structural system. Appendix F gives a summary of how Abendroth and Greimann (1989) suggest that $M_w$ is calculated.

Lateral displacement of the abutments, $\Delta_{abut}$, will result in a first order elastic moment, $M_T$, which for fully integral abutments can be calculated as

$$M_T = \frac{6 \cdot E_p I_p \cdot \Delta_{abut}}{L_{equ}^2}.$$  \hspace{1cm} (4.4)

**Alternative 2**

This alternative is unlike Alternative 1 based on inelastic behaviour. This model allows the occurrence of plastic hinges, and that lateral displacement of the top of the piles can lead to plastic redistributions of internal forces. Lateral movements of the abutments will induce stresses in the piles. The first-order plastic theory for small displacements says that support movement will not affect the plastic failure. These stresses are therefore not taken into consideration in the design of the piles. Alternative 2 is recommended for the design of piles which have a sufficient moment-rotation capacity. Abendroth and Greimann (1989) suggest that a ductility criterion, which they have developed themselves together with Johnson and Ebner, is used to determine the moment-rotation capacity. This criterion is described in Appendix D.

The axial pile force, $N$, will result in a moment, due to the displacement of the top of the pile. The moments that are induced will in this case be second order bending moments, and are for fully integral abutments calculated as,

$$M_N = \frac{N \cdot \Delta_{abut}}{2}.$$  \hspace{1cm} (4.5)

The total moment applied at the top of the pile will be the sum of the moment caused by the rotation of the abutment and the second-order bending moment due to the displacement.

$$M = M_w + M_N$$  \hspace{1cm} (4.6)
5 Bridge Temperature

A bridge is a massive structure which will be directly exposed to varying weather conditions. Thermal actions are therefore more important when bridges are designed, compared to the design of load bearing elements in buildings, which often are embedded into the structure.

Bridges will be exposed to temperature changes throughout their lifetime. The length of a bridge will be increasing and decreasing when the temperature is changing. In conventional bridges, expansion joints and bearings are used to take care of movements. An integral bridge has no moveable joints, the superstructure and the abutment forms a rigid structure. When the length of the bridge is changing, due to thermal variations, there will be lateral displacements of the abutments and the piles. When the temperature is increasing the abutments are pushed against the backfill, and when the temperature is decreasing they are pulled away from the backfill, see Figure 5:1. There will be repeating cycles of expansion and contraction throughout the bridge lifetime. (Dicleli and Albhaiisi 2004, Arsoy 2000)

![Figure 5:1 Movements of the abutments caused by temperature variations (Arsoy 1999).](image)

5.1 Factors Affecting Thermal Movements

Thermal movements of the abutments are dependent on the length of the bridge, the material in the superstructure, and variations in bridge temperature due to different weather conditions.

5.1.1 Weather Conditions

The bridge temperature and its behaviour differ from one location to another. The climate at the specific location is governing the thermal variations in the structure. It is hard to understand and describe how meteorological conditions are affecting a bridge, but the most governing factors have been identified by England et al. (2000) and are also described by Arsoy (2000).
The shade temperature of the air, which is surrounding the bridge construction, is of course one of the most important factors that are affecting the bridge temperature. The shade air temperature should be measured in a standardised way, which ensures that the measured temperatures are not affected by solar radiation, wind speed, precipitation etc.

Higher solar radiation will give higher bridge temperature, if the other parameters that are influencing the bridge temperature are kept constant. Solar radiation is varying over the year in most of the world, due to varying approach angles into the atmosphere. It is also varying daily, with the peak value in the middle of the day and the lowest value in the night. The solar radiation can also be shifting from one minute to another, due to clouds.

The wind speed affects the bridge temperature by transporting air with different temperatures from one place to another. This gives a faster heat transfer between air and structure. In general, high wind speeds will give lower bridge temperatures.

The temperature in a structure will be affected by rain and snow that are falling on it. Heat transfer will take place when the precipitation comes into contact with the surface of the structure. Heat will be transferred from the warmer part to the cooler part. The bridge will in most of the cases be warmer than the precipitation. Therefore, the bridge temperature will in general be reduced by precipitation.

The materials that are used in the bridge are also affecting the temperature. The air temperature can change rapidly, but structural materials will not immediately adjust their temperature to the air temperature. If the temperature is measured inside a material, a damping effect of the temperature changes should be observed. How fast a material adjusts its temperature to the surroundings depends on the thermal properties of the material, thickness, area that is exposed to the air, and the different weather conditions mentioned above.

Steel details will adjust their temperature faster than concrete details, while steel has a higher thermal conductivity than concrete, 40-60 W/m·K vs. 1-2 W/m·K (Cengel 1997). A bridge girder made of steel is also much thinner than any part of the bridge that is made of concrete. The steel parts will therefore adjust their temperature even faster in comparison with concrete parts. The coefficient of expansion is the same for both steel and concrete, \( \alpha = 1.0 \times 10^{-5} \, ^\circ\text{C}^{-1} \), according to the Swedish Bridge Code – BRO2004 (Vägverket 2004). Yet, bridges made of steel will have larger movements than bridges made of concrete, while steel faster adjusts its temperature to the surrounding. According to research made by Emerson (1980, 1982), concrete bridges have effective bridge temperatures (EBT) that are almost the same as the average air temperatures over the previous two days. The EBT of bridges with composite superstructures are more related to the temperature of the last 24 hours. Different types of superstructures will give different patterns of cyclic displacements of the abutments. The cyclic loading of the supporting piles will therefore be dependent on how the superstructure is designed. The seasonal movements of a composite bridge deck are in general about 121 % of the movements of a similar bridge with concrete deck (England et al. 2000). The load-path history for the piles will be complex and unique to each bridge type, length, and location.


5.2 Thermal Distribution

There are a lot of factors that are affecting the temperature of a bridge, and all factors have their own way of influencing the bridge temperature. Arsoy (2000) describes a temperature distribution model, created by Mary Emerson. This model shows how the bridge superstructure is affected over the depth. The temperature in the upper part is mostly dependent on solar radiation and the temperature in the lower part is more dependent on the shade air temperature and the heat transfer from the ground. The middle part of a superstructure’s cross-section, especially one made of concrete, will mainly be affected by the weather conditions in the past few days. Figure 5:2 illustrates how Emerson describes the temperature distribution.

![Figure 5:2 Illustration of how different factors influencing the temperature distribution in a bridge superstructure (Emerson 1977, see Arsoy 2000).](image)

The varying temperature distribution, described above, is not easy to use in calculations. The temperature distribution in the superstructure is therefore separated in two parts, the mean temperature and a thermal gradient. Lateral thermal movements of the abutments are primary caused by the varying mean temperature of the bridge, and thermal gradients are mainly responsible for the bending moments in the superstructure caused by a non uniform vertical temperature distribution. (Arsoy 2000)

A non-linear temperature profile through the dept of the bridge will give stresses in the superstructure as well as in the piles in integral abutment bridges. This must be taken into consideration when bridges are designed. The proposal to Eurocode, ENV 1991-2-5, separates the complex temperature distribution into three different components, see Figure 5:3. (Soukhov 2000)

![Figure 5:3 The non-linear temperature and its components according to ENV 1991-2-5 (Soukhov 2000).](image)
(a) - Uniform temperature component, $\Delta T_N$
This component is calculated as the bridge effective temperature, $T_N$, subtracted by the datum temperature, $T_0$. The datum temperature is the temperature at the day when the top of the abutments is cast, and the rigid connection between abutments and girders is formed. If the datum temperature is unknown it is recommended that $10^\circ C$ is used as $T_0$. The uniform temperature component will mainly be responsible to the lateral movements of the abutments.

$$\Delta T_N = T_N - T_0$$  \hspace{1cm} (5.1)

(b) - Linearly varying temperature component, $\Delta T_M$
This temperature component will cause bending moments in the superstructure and rotations of the abutments, as a result of the rigid connection between girders and abutments. It is varying linearly though the dept of the superstructure, and can be separated into a vertical and horizontal part. In general, only the vertical part is taken into consideration for bridges.

(c) - Non-linear temperature distribution, $\Delta T_E$
The part of the temperature distribution which is non-linear will create internal stresses which also are non-linear. These stresses will be in self equilibrium over the cross-section of a bridge, and will therefore not contribute to any load effect in the piles.

5.3 Effective Bridge Temperature Calculations
Longitudinal thermal movements of a bridge are mainly governed by the effective bridge temperature (EBT), $T_N$. The EBT is a complex parameter which is hard to determine. Several factors are influencing the EBT, as the materials in the superstructure, geometry, air temperature, solar radiation, wind speed, and precipitation (see Chapter 5.1). The weather related factors are not easy to predict and they are also highly variable. Another problem is the relationship between the weather parameters, it is hard to measure or decide how big the contribution is from one certain factor at a specific time.

Mary Emerson created, in the 1970’s, a model where the EBT is related to the shade air temperature. Her tests and theories have later been reviewed in other research projects. Oesterle and Volz (2005) have for instance tried to evaluate Emerson’s theories and adjust them to a more varying climate than the climate on the British islands. They have made some statistical analyses of the relationship between the shade air temperature and the EBT. They came to the conclusion that there are only small differences between the shade air temperatures in the last 24 respectively 48 hours. The old model uses the mean temperature during the last 48 hours for concrete superstructures. The results from Oesterle and Volz analyze indicate that the mean temperature could be taken during either 24 or 48 hours, independent of the type of bridge superstructure. Their study resulted in a new way of determining the mean temperature during the last 24 hours by using the minimum and maximum shade air temperatures, taken from tables. The following equations where defined by Oesterle and Volz (2005).
Bridge Temperature

\[ T_{\text{min\ EBT}} = 1.0 \cdot T_{\text{min\ shade}} + 5^\circ C \quad (5.2) \]
\[ T_{\text{max\ EBT}} = 2.2^\circ C + \Delta T_{\text{solar}} \quad (5.3) \]

\textbf{Composite bridges}
\[ T_{\text{min\ EBT}} = 1.04 \cdot T_{\text{min\ shade}} + 2.4^\circ C \quad (5.4) \]
\[ T_{\text{max\ EBT}} = 1.09 \cdot T_{\text{max\ shade}} - 0.1^\circ C + \Delta T_{\text{solar}} \quad (5.5) \]

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T_{\text{min\ EBT}} )</td>
<td>minimum effective bridge temperature</td>
</tr>
<tr>
<td>( T_{\text{max\ EBT}} )</td>
<td>maximum effective bridge temperature</td>
</tr>
<tr>
<td>( T_{\text{min\ shade}} )</td>
<td>minimum shade temperature at the bridge location</td>
</tr>
<tr>
<td>( T_{\text{max\ shade}} )</td>
<td>maximum shade temperature at the bridge location</td>
</tr>
<tr>
<td>( \Delta T_{\text{solar}} )</td>
<td>uniform temperature change from direct solar radiation</td>
</tr>
<tr>
<td>( T_1 )</td>
<td>solar incremental temperature</td>
</tr>
</tbody>
</table>

Oesterle and Volz (2005) derived an expression for \( \Delta T_{\text{solar}} \), from their studies on concrete and composite bridges. The bridge temperature change due to solar radiation was measured as a function of the solar incremental temperature, \( T_1 \), at the top of the bridge deck. Field tests showed that \( \Delta T_{\text{solar}} \) was varying between 0.18-0.28 \( T_1 \) for concrete bridges and between 0.14-0.19 \( T_1 \) for composite bridges. Oesterle and Volz (2005) proposed 0.26\( T_1 \) and 0.18\( T_1 \) as design values for concrete respectively composite bridges.

### 5.4 Design Codes for Thermal Actions

This section is a summary of how Eurocode and the Swedish Bridge Code are dealing with the thermal actions on a bridge structure.

#### 5.4.1 Eurocode

The work with the development of a European standard, Eurocode, has resulted in some proposals for design rules for thermal actions on bridges. Eurocode is still in the phase of development, and different countries prefer different approaches. This section is based on a proposal for EN1991-1-5 (Soukhov 2000), and there might be some differences to the final EN1991-1-5 document.

In order to deal with different materials and their different properties, three different groups of superstructures are considered in Eurocode for thermal actions.

\begin{itemize}
  \item \textbf{Group 1: Steel deck on steel box, truss or plate girders}
  \item \textbf{Group 2: Concrete deck on steel box, truss or plate girders}
  \item \textbf{Group 3: Concrete slab or concrete deck on concrete beams or box girders}
\end{itemize}
The effective bridge temperatures (EBT) are derived for each group of superstructures. The real situation, with influence from solar radiation, precipitation, wind etc, has been simplified into a model where the shade air temperature alone is used to estimate the EBT. This simplification is not so far from the truth, Soukhov (2000) states that the EBT mainly is governed by the shade air temperature, with some influence from the average value of solar radiation. He also states that solar radiation is the dominant variable for the linear temperature distribution, which is taken into consideration by the temperature gradient. Mary Emerson has made a lot of research work, back in 1970’s, about how the temperature affects bridges. Her research has been the background for the development of the following graphs, which can be found in the proposal for EN1991-1-5 (Soukhov 2000), see Figure 5:4. The graphs show how maximum and minimum values of the shade air temperature can be used in order to estimate the EBT for different type of bridges.

![Figure 5:4](image)

**Figure 5:4** Correlation between max/min shade air temperature and max/min EBT.

The second component in the temperature distribution is the linearly varying temperature component. This component is taken into consideration by using characteristic values for different types of bridges. The characteristic values are results of statistical analyses, which have given the following values, see Table 5:1. These values occur with a probability of one time in 50 years.

<table>
<thead>
<tr>
<th>Type of superstructure</th>
<th>Positive temperature difference $\Delta T_{\text{Mpos}}$ [°C]</th>
<th>Negative temperature difference $\Delta T_{\text{Mneg}}$ [°C]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel deck on steel box, truss or plate girders</td>
<td>18</td>
<td>-13</td>
</tr>
<tr>
<td>Group 2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete deck on steel box, truss or plate girders</td>
<td>15</td>
<td>-18</td>
</tr>
<tr>
<td>Group 3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete deck on:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- concrete box girder</td>
<td>10</td>
<td>-5</td>
</tr>
<tr>
<td>- concrete T-girder</td>
<td>15</td>
<td>-8</td>
</tr>
<tr>
<td>- concrete slab</td>
<td>15</td>
<td>-8</td>
</tr>
</tbody>
</table>
Positive temperature difference means that the top surface of the bridge superstructure is warmer than the bottom surface. Negative temperature difference is the opposite situation with a warmer bottom surface. The positive temperature difference for the linear temperature distribution will be largest during days with high solar radiation (Soukhov 2001).

Eurocode 1991-1 defines some other temperature values as functions of the characteristic temperatures. These values are calculated by adding reduction factors to the characteristic values. The Eurocode proposal documents give the following reduction factors for thermal actions on bridges according to Soukhov (2001), see Table 5:2.

<table>
<thead>
<tr>
<th>Infrequent value (once in a year)</th>
<th>$\Delta T_1 = \psi_1 \cdot \Delta T_k$</th>
<th>$\psi_1 = 0.8$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frequent value (once in two weeks)</td>
<td>$\Delta T_2 = \psi_2 \cdot \Delta T_k$</td>
<td>$\psi_2 = 0.5$</td>
</tr>
</tbody>
</table>

The non-linear temperature distribution, the third component (c) presented in Chapter 5.2, is not used in the design of bridges in most of the countries in Europe. Therefore, it is not included in the main part of the proposal for Eurocode 1991-1-5. United Kingdom is the only country that is using the non-linear temperature distribution for bridge design. Their way of using it is described in an annex to ENV 1991-2-5, see Appendix A.

5.4.2 Swedish Bridge Code – BRO2004
The Swedish Bridge Code for thermal actions is based on the variations of the uniform temperature, and shall be combined with the worst alternative of three different non-uniform temperature distributions.

*Uniform temperature changes*
The uniform temperature change shall be assumed to be uniform in the entire superstructure. The temperatures that are used in the design calculations are the extreme values of the mean temperature in the construction. How these temperatures are calculated is shown in Table 5:3, which is redrawn from BRO2004.

<table>
<thead>
<tr>
<th>Type of superstructure</th>
<th>Mean bridge temperature $[^{°C}]$</th>
<th>Temperature differences $[^{°C}]$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$T_+$</td>
<td>$T_-$</td>
</tr>
<tr>
<td>Steel or aluminum deck on steel box girders or steel I-girders</td>
<td>$T_{\text{max}} + 15$</td>
<td>$T_{\text{min}} - 5$</td>
</tr>
<tr>
<td>Concrete or wood deck on steel box girders or steel I-girders</td>
<td>$T_{\text{max}} + 5$</td>
<td>$T_{\text{min}} - 5$</td>
</tr>
<tr>
<td>Concrete deck on concrete box girders, T-girders or slabs.</td>
<td>$T_{\text{max}}$</td>
<td>$T_{\text{min}} + 10$</td>
</tr>
<tr>
<td>Wood deck on wood girders</td>
<td>$T_{\text{max}} - 5$</td>
<td>$T_{\text{min}} + 10$</td>
</tr>
</tbody>
</table>
\( T_{\text{max}} \) and \( T_{\text{min}} \), in Table 5:3, are the maximum and minimum shade air temperatures that occur with a probability of one time in fifty years. These values are given for different locations in Sweden by two maps with isotherms, see Appendix E.

Non-uniform temperature changes

1. The non-uniform temperature changes in a cross-section, \( \Delta T \), are assumed to be linearly distributed and with a zero point that coincides with the cross-section’s center of gravity. The magnitudes of the differences are taken from Table 5:3.

2. If different elements of a bridge structure are exposed to different temperatures, then it shall be assumed that there is a temporary temperature difference between the elements, horizontally as well as vertically. If at least one of the elements is made of concrete, then the temperature difference shall be taken as 10°C, in other cases it is assumed to be 20°C.

3. If concrete box cross-sections are used in a bridge, then it shall be assumed that a temperature difference of 15°C can occur between the inner and outer wall of the box cross-section.

5.5 Long Term and Short Term Temperature Changes

There will be both daily and seasonal temperature changes which will affect the bridge. The daily variation in temperature between night and day creates a cycle of expansion and contraction. In the general case, this cycle is repeated every day. There is also a seasonal change in temperature. The mean temperature reaches the highest level in the summer, and the lowest temperature is observed during the winter. The largest expansion will therefore occur during summer days and the largest contractions can be observed during winter nights. (Arsoy 1999)

The daily variations in temperature will create temperature cycles with low amplitude and high frequency. The seasonal changes will on the other hand give annual temperature cycles with high amplitude and low frequency. If a mathematical model of the temperature is created, the seasonal changes could be described by a modified sinus curve, see Figure 5:7. The daily temperature changes could be superposed on the annual sinus curve. In many cases the daily changes can be disregarded, while the variations are small compared to the seasonal changes. Another reason to neglect daily temperature changes is the fact that there is a thermal inertia in the material which will give a damping effect on the short term changes. This report is focused on pile fatigue, and daily temperature movements will give a contribution to the fatigue. Hence, daily changes are not neglected.

Russel and Gerken (1994) noted that the annual temperature variations mainly affect the changes in the length of the superstructure, while the daily temperature changes mainly affect the vertical temperature gradient in the superstructure. A field study made in Iowa was performed on integral bridges with steel and concrete girders. This study showed that bridge length and pile strains are varying due to daily temperature variations (Girton et al. 1991). The lateral movements caused by the daily temperature changes should therefore be taken into consideration in the fatigue calculations.
5.5.1 Estimating Thermal Displacements

A conservative estimation of thermal movements can be made by using the theoretical variations in the length of the superstructure, due to changes in temperature. Equation (5.6) describes how the length of the bridge, \( L_b \), is changing due to changes in temperature. Changes in length, \( \Delta L \), are directly proportional to changes in temperature, \( \Delta T_N \), with the thermal coefficient, \( \alpha \), as proportional constant.

\[
\Delta L = \alpha \cdot \Delta T_N \cdot L_b
\]  
(5.6)

\( \Delta T_N \) is the difference between the temperature at the construction day and the effective bridge temperature at a certain time. It is important that the designer of the bridge take into consideration the maximum variations in temperature, \( \Delta T_{\text{max}} \), while this value is governing how large movements and stresses the abutments and piles will be experiencing.

A field study (Frosch et al. 2005) made in Indiana, shows that the actual lateral displacements of the abutments are a bit smaller than the theoretical displacements. The differences can be explained by resistance from piles, restraint from the backfill, and approach slab friction. Figure 5:5 illustrates the differences between calculated and measured movements. The bridge where the measurements took place is a 112 m five-span integral abutment bridge, with a superstructure made of pre-stressed concrete girders. The bridge is a crossing over the Mississinewa River on State Road 18, located in Indiana (USA).

![Figure 5:5 Comparison between theoretical and measured movements (Frosch et al. 2005).](image)

The same study (Frosch et al. 2005), lead to the conclusion that movements of the abutments, caused by thermal expansion and contraction of the superstructure, are primary horizontal sliding movements. It is noted that small rotations of the abutments also occur, these are however so small that the authors suggest that they should be ignored in an analysis. This observation by Frosch et al. is not proved to be general, and might be a result of a stiff superstructure, a short end span or something else. Rotations of the abutments are not ignored in the calculations in this report, since even small rotations can be important in the fatigue calculations.
One of the conclusions from a study made by Dicleli and Albhaisi (2003) is that, since concrete bridges are less sensitive to temperature variations they are better suited for integral constructions than composite bridges. This is general for all integral bridges, but when the climate gets colder the recommendation to use concrete bridges become even stronger. Calculations according to the Swedish Bridge Code, made by Pétursson (2000), shows that a bridge in northern Sweden will move 0.82 mm/length meter if it is a composite bridge, and 0.57 mm/length meter if it is a concrete bridge. In Tennessee (USA) bridges are designed for movements that are calculated to be 0.78 mm/length meter for a composite bridge and 0.42 mm/length meter for a concrete bridge. The bridge temperature range that are used in Tennessee is -18°C to 49°C for steel bridges, and -7°C to 32°C for concrete bridges (Abendroth and Greimann 2005).

Thermal movements are limiting how long an integral bridge can be. Although, there are no guidelines today to determine how long an integral bridge can be. Bridge engineers in the USA seem to depend on their experiences from previously constructed integral bridges when they are determining the maximum length of a new one. A guideline to determine the maximum length is needed, and there have been studies made in this area in the past few years. (Dicleli and Albhaisi 2003)

5.6 Thermal Effects on Piles

It is important to predict thermal movements while the stresses in the piles will be directly dependent on these. The abutment piles can be subjected to large bending moments, caused by the contraction and expansion of the bridge superstructure. The vertical load capacity of the piles can be reduced as a result of the thermal movements (Bayoglu Flener 2004).

Lateral abutment displacements are not the only effect from temperature changes that affect integral abutment piles. Vertical temperature gradients will occur in the bridge superstructure, and if the temperature distribution curve does not coincide with the rotation center of the girders then a secondary bending moment will occur (Arsoy 1999). This varying moment will give rotations of the abutments and will therefore also affect the piles.

The bridge superstructure will expand and contract as a result of seasonal temperature changes. These seasonal changes of the bridge length will cause an annual cycle of lateral displacements of the top of the piles. There will also be a lot of smaller pile displacements caused by daily or weekly temperature changes, and by daily thermal gradients in the superstructure. Thermal gradients are smaller in the winter compared to the summer. The minor cycles will therefore appear more frequently during summer than in winter, see Figure 5:6. But the amplitudes of daily/weekly changes are often higher in the winter. A sharp change in temperature is often the reason to the relatively large minor cycles in the winter. This behavior is confirmed by field measurements from a study performed in Iowa (Abendroth and Greimann 2005), and a research study made in England (England et al. 2000).
Data recorded from field tests shows that piles will be subjected to daily or weekly variations in strain. According to Dicleli and Albhaisi (2004), these short term changes have amplitudes of approximately 20-40% of the annual strain amplitude, $\varepsilon_a$, see Figure 5:7. Several field tests in the USA are also indicating that the daily movements have larger amplitudes in the winter than in the summer (Arsoy 2000).

England et al. (2000) point out that daily variations in EBT are smaller in the winter than in the summer, they are just 20-30% of the variations in the summer. If the abutment displacements are assumed to be directly dependent on the EBT, this would be a contradiction to the field tests in the USA. One could speculate upon if it is the British Islands that has another type of climate and therefore get larger variations in the summer, or are the pile stresses not directly dependent on the EBT? Another possibility would be that the authors refer to different behaviors. England et al. might refer to the total variations during the summer and the winter when they note that the variations are 20-30% larger in the summer. While the field test from the USA might refer to the more frequently higher amplitudes that is observed during the winter.
The temperature at the casting day will affect the zero strain level in the piles. It may not be at the point where the second order derivative, of the seasonal temperature curve, is zero. In the general case, it will be a net difference between the zero strain level and the maximum and minimum strain levels. The location of the bridge and the climate will influence how large the net differences in strain will be. In order to be able to describe the strain cycle in a proper way, Dicleli and Albhaisi (2004) defined two new amplitudes. The positive strain amplitude, $\varepsilon_{ap}$, will reach its peak during summertime, and the negative strain amplitude, $\varepsilon_{an}$, has the peak of the net strain in the wintertime, see Figure 5.7. (Dicleli and Albhaisi 2004)

When Dicleli and Albhaisi (2004) made their study of cumulative fatigue in steel H-piles, they neglected the difference between the positive and negative strain amplitudes. The explanation of why the differences can be neglected is the fact that the range of the total strain amplitude is far more important for the fatigue than the strain amplitude for single cycles.

### 5.7 Simplified Structural Analysis of Thermal Movements

The thermal contribution to the strains in integral abutment piles is assumed to be a direct result of the thermal movements. Which in turn are directly proportional to changes in EBT. Equation (5.7) gives an estimation of how large the lateral movements of the abutments will be, due to variations in bridge temperature. The expression is based on an assumption that the lateral movements have the same magnitude at both abutments. Abendroth and Greimann (1989) used this equation to determine the thermal displacement of the abutments. Their analysis is approximate and neglects the passive soil pressure and the lateral pier stiffness when the displacement of the abutments, $\Delta_{\text{abut}}$, is calculated.

$$\Delta_{\text{abut}} = \frac{\alpha \cdot \Delta T_N \cdot L_b}{2}. \quad (5.7)$$

The length of the bridge is denoted $L_b$ and the difference in effective bridge temperature between a certain day and the construction day is denoted $\Delta T_N$ (see Chapter 5.2 Thermal Distributions).

Integral abutment piles can be treated as beam-columns and designed by the cantilever method. Equivalent cantilevers with a length $L_{\text{equ}}$ are used in the calculation, see Appendix B. The lateral displacements caused by bridge temperature variations will induce a pile moment, $M_T$, and a lateral pile force, $F_T$. Abendroth and Greimann (1989) state that (5.8) and (5.9) can be used to calculate the pile moment and the lateral pile force, caused by thermal variations.

$$M_T = \frac{6E I \Delta_{\text{abut}}}{L_{\text{equ}}^2} \quad (5.8)$$

$$F_T = \frac{12E I \Delta_{\text{abut}}}{L_{\text{equ}}^3} \quad (5.9)$$
Figure 5.8 shows a free body diagram, which presents the forces and moments that are acting on an integral bridge.

The abutment wall height is denoted H, and the length of the end span is denoted \( L_{es} \). When the bridge temperature increases and the superstructure becomes longer, the abutment will be pushed against the backfill behind the abutment wall. The passive soil pressure from the backfill is represented by a horizontal force, \( F_p \). This force will act on a distance \( H_p \) from the top of the abutment wall, and can be estimated by using a soil model, for instance Rankine’s theory of earth pressure, see Appendix C.

\[
F_p = \frac{\gamma \cdot H^2 \left[ 1 + \sin \phi \right]}{2 \left[ 1 - \sin \phi \right]}. \tag{5.10}
\]

Thermal movements will also give rise to axial forces in the piles, \( N_T \), in order to keep the force situation in equilibrium. This axial force can be calculated by an equilibrium equation, which can be derived from the free body diagram in Figure 5.8.

\[
N_T = \frac{(F_p \cdot H_p) + (F_T \cdot H) + M_T}{L_{es}} \tag{5.11}
\]

### 5.8 Measuring Bridge Average Temperature

If temperature measurements are made on an existing bridge, it would be possible calculate the average bridge temperature, \( T_{ave} \), and the resulting thermal movements. Bridge superstructures are often constructed with different materials and different cross-sections, and should therefore be divided into \( n \) segments in an analysis of the temperature. Temperature measurements should be performed in every segment. If the temperature distribution \( (\Delta T_j) \) inside of segment \( j \) with the area \( A_j \) is approximated as uniform, then the average bridge temperature can be calculated as a weighted average of the segments temperatures, see (5.12).

\[
T_{ave} = \frac{\sum_{j=1}^{n} \Delta T_j A_j}{\sum_{j=1}^{n} A_j E_j} \tag{5.12}
\]
Girton et al. (1991) took this equation a step further and used it to calculate the longitudinal displacement of a bridge, $\Delta_b$, due to thermal movements. The unrestrained displacements of each segment in the superstructure, $\Delta_j$, are used to calculate a weighted average displacement of the bridge. To be able to do that, a uniform coefficient of thermal expansion, $\alpha_j$, is introduced for every segment, and also a uniform modulus of elasticity, $E_j$. Girton et al. suggest that bridge displacement is calculated as

$$
\Delta_b = \frac{\sum_{j=1}^n \alpha_j \Delta_j A_j E_j}{\sum_{j=1}^n A_j E_j} \cdot L,
$$

where $L$ is the length of all segments.

### 5.9 Available Temperature Measurements

The most proper way of studying temperature changes in a bridge, would be by performing field measurements in a real bridge superstructure. This could be done by embedded thermal sensors that are mounted at different depths of a cross-section. But if there are no real temperature measurements available, estimations can be done by using shade air temperature.

The Swedish Meteorological and Hydrological Institute (SMHI) are measuring the shade air temperature at hundreds of stations spread out over the whole county. Some of the stations are manned but most of them are automatic nowadays. They have been recording the mean temperature of each month from the start of the measurements, in some cases from the 18th century. Daily temperature measurements are recorded and available in their database, from 1961 until today. Some stations are recording the temperature every hour, and others are recording it at least three times a day. (SMHI 2006)

This database can be a good source in order to find out how the shade temperature is varying at a location where a bridge shall be constructed.

### 5.9.1 Measurement Equipment

In order to measure the shade air temperature and nothing else, thermometers have to be protected from solar radiation and precipitation in some way. Different techniques have been used through the years. For instance, white painted cages made of wood, or smaller radiation shelters made of white painted plastic or metallic materials. The thermometers are in general placed 1.5 m above the ground surface and with no high surrounding objects closer than their height. The thermometers that are used are often resistance thermometers which automatically are recording the temperature at certain times. Some mercury thermometers are also used, those are not so common nowadays but they have been used a lot during the period since 1961. Hence the temperature database contains a lot of temperatures measured by those. (SMHI 2006)
5.9.2 Sources of Errors

There will always be some errors when the temperature is measured during a long period, mostly not because of the thermometers themselves. The thermometers have in general good precision, and do not measure more than a few tenth of a degree wrong. The sources of errors are instead often the lack of protection against solar radiation, and bad placing of thermometers. As a result of this, to high shade temperatures might sometimes be recorded. Fast changes in temperature can also be a source to misleading temperatures. (SMHI 2006)
Low-cycle Fatigue of Steel Piles in Integral Abutment Bridges
6 Low-cycle Fatigue

Low-cycle fatigue is fatigue caused by strain cycles involving plastic deformations. Large temperature displacements and cyclic loading of integral abutment piles can cause plastic deformations and lead to low-cycle fatigue failures. Thermal movements of the abutments can give elastic as well as plastic deformations of the piles, depending on how long the bridge is and how large the variations in EBT are. Rotations of the abutments due to varying traffic loads and temperature gradients will also cause deformations of the piles. These deformations must also be taken into account in an analysis of a possible low-cycle fatigue failure.

The majority of the states in the USA, and among other countries as well, are orienting their H-piles for weak axis bending in integral abutment bridges. Because of that, the following section is focused on low-cycle failure for H-piles oriented in weak axis bending.

Lateral displacements at the top of the piles will lead to varying stresses in the flanges of the H-piles, oriented for weak axis bending. It is possible that these stresses will exceed the yield strength now and then during the bridge lifetime. The tip of the flanges will then yield and plastic deformations take place. How frequently plastic deformations might occur depend on climate, soil properties, bridge length, pile cross-section etc. The low-cycle fatigue failure will start with small cracks that appear at the tip of the flanges. These cracks will propagate towards the web under further cyclic loading, see Figure 6:1. The width of the flanges that can transfer axial loads become smaller and smaller, and the web has to take more and more axial load. The part of the pile where the cracks are propagating starts to work more as a hinge, until an ultimate failure of the web take place. (Huang et al. 2004)

One way of estimating the time until a low-cycle fatigue failure is to use a strain-based approach. In this approach, the number of displacement cycles that a structural unit can withstand is formulated as a function of the plastic strains in the studied area of a structural member. Piles in integral abutments will be subjected to lateral movements that in many cases are quite large and plastic deformations are expected. Dicleli and Albhaiisi (2004) state that, a strain-based approach is an appropriate way of estimating the number of cycles until low-cycle fatigue failure for steel piles in integral abutment bridges.

Figure 6:1 Illustration of low-cycle fatigue failure in a steel H-pile (Huang et al. 2004).


6.1 Low-cycle Fatigue Prediction

Huang et al. (2004) presents three different methods which can be used to predict low-cycle fatigue failures. The methods are: General Strain Life Equation, Manson’s Universal Slope Equation, and extrapolated strain-cycle curves.

Dicleli and Albhaisi (2004) are also presenting a strain based approach for calculations of cumulative fatigue. This approach is a simplified version of the general strain life equation and especially made for H-piles.

6.1.1 General Strain Life Equation

The general strain life equation is a strain based approach to fatigue problems. The total strain in an element is the sum of the elastic and plastic strains. The magnitude of the strain will govern which strain component that will be dominant. At small strains, elastic strains will be dominant and the strength will control the performance. At large strains, plastic strains will be dominant and the ductility of the steel will control the performance. The perfect material would be one with high strength and high ductility. A perfect material is not available today, and the designers of the structures have to make compromises between strength and ductility depending on the loads and strains that are expected. (ASM Handbook 1996)

Figure 6.2 illustrates the strain situation in an element. The elastic and plastic strain components are both represented by straight lines in a log-log diagram.

Figure 6.2 Strain-life curves showing the total-, elastic-, and plastic strain components (ASM Handbook 1996).

The general strain life equation for small smooth axial specimens is given in (6.1). This expression is based on an assumption that the displacement amplitude is totally reversed under each cycle. This might not be the case in the reality, but this model gives an answer that will be on the safe side.
\[
\frac{\Delta \varepsilon}{2} = \frac{\sigma'_f}{E} (2N_f)^c + \varepsilon'_f (2N_f)^b
\]  
(6.1)

- \( \Delta \varepsilon/2 \)  total strain amplitude
- \( N_f \)  fatigue life
- \( \varepsilon'_f \)  fatigue ductility coefficient
- \( \sigma'_f \)  fatigue strength coefficient
- \( c \)  fatigue ductility exponent
- \( b \)  fatigue strength exponent

### 6.1.2 Coffin-Manson’s Universal Slope Equation

Coffin-Manson’s equation is a simplified form of the general strain life equation. This model is more conservative and gives higher safety margins than the previous model. It has been developed from tensile tests on small specimens. The simplified equation is written as

\[
\Delta \varepsilon = 3.5 \frac{\sigma_u}{E} (N_f)^{0.12} + \varepsilon'_f (N_f)^{0.6}.
\]  
(6.2)

- \( \Delta \varepsilon \)  total strain range
- \( \varepsilon_f \)  elongation at fracture
- \( \sigma_u \)  ultimate tensile strength

Figure 6:3 illustrates an example of the Coffin-Manson’s equation, assuming a steel with ultimate strength of 500 MPa and an elongation at fracture that is 20 %.

### 6.1.3 Extrapolated \( \Delta \varepsilon - N_f \) Curves

Another way of evaluating the low-cycle fatigue would be by using the \( f_{ch} - N_f \) (characteristic fatigue strength – number of stress cycles until failure) curves given in the Swedish Regulations for Steel Structures - BSK99. The curves that can be found in BSK99 have to be extrapolated since they are illustrated only for \( N_f \) larger than 1000. The curves are extrapolated to one cycle and also converted from stresses into strains. The expressions for the \( f_{ch} - N_f \) curves can be rewritten as
Low-cycle Fatigue

\[ \Delta \varepsilon = \frac{C}{E} \left( \frac{2 \cdot 10^6}{N_f} \right)^{1/3} \quad \text{if} \quad N_f \leq 5 \cdot 10^6 \tag{6.3} \]

\[ \Delta \varepsilon = 0.885 \cdot \frac{C}{E} \left( \frac{2 \cdot 10^6}{N_f} \right)^{1/5} \quad \text{if} \quad 5 \cdot 10^6 < N_f < 10^8 \tag{6.4} \]

where \( C \) is the detail fatigue category factor, which is defined as the characteristic fatigue strength at \( 2 \cdot 10^6 \) stress cycles with constant amplitude. The \( C \)-factor equals 112 MPa, for a rolled cross section of performance class GB, according to BSK99. The \( f_{ik} - N_f \) curves given in BSK99 have two different levels for the fatigue endurance limit for steel. Which curve that should be used depends on in which extension the constructions will be affected by corrosion. If the construction has a well maintained corrosion protection, then it can be presumed that the endurance limit occurs at the strain giving \( 5 \cdot 10^6 \) cycles until failure. For other constructions, it is presumed that the endurance limit occurs at the strain giving \( N_f = 10^8 \) cycles. Figure 6:4 illustrates the \( \Delta \varepsilon - N_f \) curves from BSK99 for a cross-section with the detail fatigue category factor \( C = 112 \) MPa. Coffin-Manson’s equation is also plotted in Figure 6:4, in order to illustrate the differences between the models.

Huang et al. (2004) refers to some tests which indicates that the \( \Delta \varepsilon - N_f \) curves gives a lower bound for low-cycle fatigue failure. It actually seems to give rather conservative answers. Coffin-Manson’s equation is on the other hand developed from tensile test results on small specimens. There is no scale factor in the equation, but it might be a scale factor in the reality. Since a larger cross-section increases the probability of the occurrence of any type of imperfection. Yet, both models are still treated as conservative in the literature.
6.2 Predicting the Level of Cumulative Fatigue

Dicleli and Albhaisi (2004) state that, it is possible to make an estimation of how many strain cycles, with constant amplitude, that will take place before it ends up in a low-cycle fatigue failure. The following expression is proposed in order to get a slight idea of when a fatigue failure will happen. Their expression is valid for steel H-piles, and it is based on the total strain amplitude, $\varepsilon_a$. The number of strain cycles until failure, $N_f$, can be expressed as

$$N_f = \frac{1}{2} \left( \frac{\varepsilon_a}{M} \right)^{m-1} \quad \text{or} \quad \varepsilon_a = M \cdot (2N_f)^m,$$

(6.5)

where $M$ and $m$ are constants that have been derived for H-piles, $M = 0.0795$ and $m = -0.448$. This formula is only valid for strain cycles with constant amplitudes. It could therefore be useful to predict the fatigue caused by the seasonal thermal movements in integral bridges, since the seasonal movements would not vary too much from one year to another. An integral bridge is however not only subjected to seasonal thermal movements. There are also movements that take place daily or weekly, as a result of different temperatures during day and night, and different weather conditions. The question is how these movements shall be taken into account when the level of cumulative fatigue is calculated. Stresses in piles caused by daily or weekly temperature variations may not be large enough to cause yielding by their own. But, if the seasonal thermal movements already have caused yielding, then the strain cycles with small amplitudes will add an extra amount of plastic deformation. Dicleli and Albhaisi (2004) state that many of the small strain cycles will occur when piles already have yielded, and therefore they must be a part of the low-cycle fatigue calculation.

The left part of Figure 6:5 shows a zoomed portion of a curve illustrating the strain-time behavior in integral abutment piles. Stresses due to the seasonal variations in temperature are so large that yielding already has taken place. A change in temperature over a day or a week can make the yielding even bigger or lowering the stress in the structure. The stress-strain behavior that takes place at the same time as the strain is changing is illustrated in the right part of Figure 6:5. Elastic perfectly plastic behavior is assumed.

![Figure 6:5 Illustration of the loading and unloading of the piles that cause small strain cycles, after Dicleli and Albhaisi (2003).](image-url)
A model used to estimate the cumulative low-cycle fatigue must be able to consider both the small and the large strain cycles. Palmgren and Miner (1924, 1945) have formulated an expression of how the cumulative fatigue damage in a structure can be calculated:

\[
\sum_{i=1}^{n} \frac{n_i}{N_{fi}} \leq 1. \tag{6.6}
\]

\(N_{fi}\) is the number of cycles until failure for a load or displacement with certain amplitude, and \(n_i\) is the number of times that a cycle with the same amplitude is repeated. This equation can be used to calculate how much of a structure’s fatigue life that has been consumed. In the case with piles in integral abutment bridges, the expression can be written in a slightly different way:

\[
\sum_{i=1}^{n} \frac{n_{di}}{N_{f_{di}}} + \sum_{j=1}^{n} \frac{n_{sj}}{N_{f_{sj}}} \leq 1 \tag{6.7}
\]

where \(n_{di}\) is the number of small cycles with a certain amplitude, which the structure has been exposed to. And \(N_{f_{di}}\) is the number of small cycles until failure for the load or displacement with the certain amplitude. \(N_{f_{sj}}\) and \(n_{sj}\) are defined in the same way, but they refer in this case to the large cycles which are a result of the seasonal variations in temperature. If the seasonal temperature cycles are assumed to have the same amplitude from one year to another, then the expression can be simplified as following:

\[
\left( \sum_{i=1}^{n} \frac{n_{di}}{N_{f_{di}}} \right) + \frac{n_{sj}}{N_{f_{sj}}} \leq 1. \tag{6.8}
\]

This simplification implies that changes in seasonal temperature cycles between one year and another are not taken into consideration. Even if the trend right now might be towards higher and higher temperatures, caused by the greenhouse effect and other phenomena, it would be hard to predict how large influences these phenomena might have in the future.

The cumulative damage could be predicted in a more simplified and conservative way, or by a model that is based on field measurements.

### 6.2.1 Simplified Cumulative Damage Model

The large strain cycles will have periods of one year. Therefore, \(n_s\) will be equal to the number of years, \(n\), that the bridge will be in service. In this simplified approach the small strain cycles could be assumed to be repeated once a week, according to strain measurements studied by Dicleli and Albahaisi (2004). The number of small cycles, \(n_{di}\), will then be 52 times the number of years, \(n\). If the seasonal strain amplitude is known as well as the constants \(M\) and \(m\), which are provided by Dicleli and Albahaisi for H-pile cross-sections. Then it will be possible to calculate the cumulative low-cycle damage. Measurements made on piles in integral bridges have indicated that strain amplitudes of daily or weekly changes are approximately in the range between 20-40% of the amplitude that is observed as a result of the seasonal changes. A constant, \(\beta\), is introduced in order to simplify the equations. This constant describes the relationship between the daily/weekly amplitudes and the seasonal...
strain amplitudes, $\varepsilon_{ad} = \beta \cdot \varepsilon_{as}$. The constant $\beta$ will have a value between one and zero, and measurements indicates a value between 0.2 and 0.4. If this constant is used and (6.5) is substituted into (6.8), then the following expression is obtained for the cumulative low-cycle fatigue damage:

$$\frac{104 \cdot n}{(\beta \cdot \varepsilon_{as})^{\frac{1}{m}}} + \frac{2 \cdot n}{(\varepsilon_{as})^{\frac{1}{m}}} \leq 1.$$  \hspace{1cm} (6.9)

### 6.2.2 Cumulative Damage Model Based on Measurements

Another approach to the cumulative low-cycle fatigue is a method that relies on results from field measurement or simulations. The principle is the same as the simplified approach, but the small changes in the strain amplitude are not treated in the same way. Instead of making an assumption that the strains will change a number of times each year, it is possible to monitor the strain in the piles or the temperature that causes the strain. Results from field tests can then be input data in order to calculate the fatigue damage from each strain cycle. The number of calculations increases dramatically when this approach is used, while the amplitudes of the small strain cycles are varying from one cycle to another. If a large amount of cycles shall be studied, some kind of computer program would be preferable for the calculations as well as a tool to identify the cycles. The large amplitude cycles from the seasonal changes in temperature can be treated in the same way as in the simplified approach. The following expression could be used in the analysis of the cumulative low-cycle fatigue:

$$\sum_{i=1}^{n} \frac{2 \cdot n_{di}}{(\varepsilon_{ada}^{m})^{\frac{1}{m}}} + \frac{2 \cdot n}{(\varepsilon_{as}^{m})^{\frac{1}{m}}} \leq 1.$$  \hspace{1cm} (6.10)

The field measurements can not go on forever because of economical reasons. Therefore, it might be necessary to perform some kind of statistical simulation based on data received from measurements. If the strains in the piles are monitored over a period of for instance 2 years, then there would be two large strain cycles and hundreds of small strain cycles that would be available for a statistical analysis. The measurements can then be used in order to create a probability distribution function, which shows how large the probability is that a strain cycle with a certain amplitude would occur. The probability function can then be one part of the input data in a simulation of pile strains during the bridge service lifetime.
6.3 Comments

In this chapter some theoretical models have been presented, which can be used in order to estimate when a fatigue failure will occur. Some of these models are widely used today, but it must be point out that they are not that accurate. In the ASM Handbook (1996) the authors consider if it is possible to aim at a model that actually gives reliable answers, so that experimental investigations would not be necessary. Today, Palmgren-Miner’s model is often used in an early stage to estimate the fatigue lifetime when a structure with variable amplitude loading is studied. Unfortunately, the model is not reliable in all cases. Especially not when strain cycles with large differences in amplitudes are studied. The following sentences are taken from the chapter Fatigue Crack Growth under Variable-Amplitude Loading in ASM Handbook Vol. 19 (1996).

“A warning must be made here: the Miner rule is fully unreliable for comparing the severity of different load spectra. As a simple illustration, compare a load spectrum to a modification of that spectrum obtained by adding a small number of high-load cycles. According to the Miner rule the addition should lead to somewhat shorter fatigue lives, whereas in general it leads to significant fatigue life improvements.”

Today, no theoretical model is able to predict the fatigue failure accurate enough. A rough estimate is the best result. Therefore, the Palmgren-Miner model is used in this report to get a slight idea of if a fatigue failure is likely to occur or not, during the bridge service lifetime. The answers from the calculations in this report can not be treated as “the truth”. Yet, they will probably give a hint whether or not a low-cycle fatigue failure is likely to happen. However, the result from all theoretical models about fatigue should be supplemented by test results.
7 Traffic Loads

Traffic loads seems to be a research area where few studies have been carried out. Among bridge designer in general, traffic loads are just assumed to be numbers given in codes. A detailed model of traffic loads, which reflects the real situation, is very difficult to create because of the randomness of the loads. (Getachew 2003)

Sundquist (1998) has compared the maximum allowable vehicle gross weight in different countries. The Scandinavian countries allow higher loads than the other European countries. There are only small differences in the allowable weight for shorter vehicles, but significant differences for long vehicles as lorries with trailer. The allowable gross weight in the USA varies from state to state, but seems to be a bit lower in comparison to the allowable weights in Europe. The reason to the higher allowable gross weights in the Nordic countries is mainly the need of timber transports.

If a traffic load model shall be created, it will be necessary to measure the real traffic load in one way or another. Field measurements can be performed over a long period of time, in order to collect a large amount of data. If there is no time or money to perform long time measurements, it might be necessary to use a sample from collected data instead. This sample can be used to generate fictitious traffic loads that represent the results from measurements over a long period of time. A Monte Carlo simulation is one way of generating the fictitious loads. This simulation technique is described in Chapter 8.

Bridge designers use traffic load models given in different codes when they design bridges. These loads are often believed to be conservative today, while they often are based on old traffic data. The vehicles are nowadays designed in another way with better damping mechanisms, an update of some of the codes might be necessary (Getachew 2003). Traffic load models in the codes are often based on the largest load that will act on the bridge during its service lifetime. This approach gives a large safety margin, perhaps larger than needed. Although, it is necessary to realize that a safety margin is needed, a balance on the edge between overestimated and underestimated design loads must be avoided. Overestimated loads will certainly give higher construction costs than necessary, but underestimated loads can lead to a collapse of the whole structure. A safety margin is needed, the question is how big. Getachew (2003) states that, the increased costs of a construction due to overestimated loads are small, and necessary to deal with the uncertainties and to make the design process as fast and simple as possible.

7.1 Traffic Fatigue Load According to BRO2004

The fatigue caused by the traffic loads must be taken into consideration when a bridge is designed. BRO2004 (Vägverket 2004) defines the traffic fatigue load as following. The fatigue load consists of two groups of loads, each group represents two axels of a vehicle, see Figure 7:1. One of the groups has axel loads of 150 kN and the distance between the axels is 1.5 m. The other group has axel loads of 180 kN and 2.0 m between the axels. The distance between the two groups of loads shall be at least 6.0 m. The axel loads shall be regarded as two point loads of 75 and 90 kN respectively. The distance between the point loads is 2.0 m. The group of loads shall be placed at the position on the bridge which gives the largest stress amplitude in the studied part of the structure. The dynamic effects of the loads have been considered when the axel loads in the code were decided.
Bridges that are trafficked by less than 10,000 vehicles per day, measured as a mean value over a year, shall be designed to withstand at least $10^5$ load cycles. The rest of the bridges shall be designed to withstand at least $4 \times 10^5$ load cycles.

### 7.2 Measured Traffic Loads

Different measurement systems have been used through the years in order to get a better understanding of the real loads that are acting on roads and bridges. The first way of measuring the vehicle weights was by using static scales. This method is very time consuming and can only be used for spot checks. A development of a rational system has been necessary, to be able to continuously weigh vehicles in motion. The Swedish National Road Administration (Vägverket) has used at least two of these measurement systems in the last two decades. The first one that was used was the Weigh-In-Motion system (WIM), which later on has been replaced by a Bridge-Weigh-In-Motion system (BWIM).

#### 7.2.1 WIM

WIM is a relatively wide term for different techniques to weigh vehicles that are traveling in high speed. Three common WIM-systems that are used nowadays are bending plate, piezoelectric sensors, and load cells. Bending plate is a strain based method, with an equipment which registers the strain in a plate that is loaded by the axle loads of the vehicles. Piezoelectric sensors are detecting the change in voltage which is caused by the load that acts on the sensor. The changes in voltage are calculated into dynamic loads which are transformed into static loads by some calibration parameters. Load cells are based on two scales which are recording the weight from the left and the right side of each axle. The recorded data from the two scales are summed up to an axle load. Axel sensors are often used in combination with these systems in order to classify the vehicles by counting the number of axels and measuring the axle distances. (McCall and Vodrazka 1997)

The WIM technology is quite old, and has been developed and refined through half a century. There were a lot of initial problems with these systems. Southgate (2000) describes the problems they had with the new WIM technology in Kentucky (USA) back in the late 60’s.
There were a lot of problems with the electronic measurements device and the calibration of it. The development of the computerized measurement devices has led to better accuracy of the measurements, but there are still problems to overcome. Getachew (2003) has studied and evaluated the results from some WIM measurements made by the Swedish National Road Administration during the time period 1993-1994. He found out that the data contained some weights and lengths of vehicles that were absurd. Before analyze was made, approximately 10% of the measured data were excluded because of the unreasonable values. Another 9% of the data was registered as vehicles with only one axel. It is obvious that the data contains such an amount of errors that it is necessary to filter the data in one way or another before an analysis is made.

7.2.2 BWIM

Bridge-Weigh-In-Motion is another way of measuring the weight of vehicles that traveling across bridges. This technique is based on strain measurements, by strain gauges that are mounted on the bridge, see Figure 7:2. A calibration of the measurement equipment is made by using a vehicle with known weight, axel space and axel load. When the behavior of the bridge under the load of the calibration vehicle is known, then the result from the strain measurements can be used to calculate weight and axel loads from any other vehicle. The Swedish National Road Administration tested this measurement technique for the first time in the summer of 2001. The test period showed that the BWIM-equipment performed well. The measurements have been extended to several bridges all over Sweden, and will continue at least until 2006. (Vägverket 2006)

![BWIM strain gauges mounted at a concrete bridge](image)

Figure 7:2 BWIM strain gauges mounted at a concrete bridge, picture from Vägverket (2003).
8 Monte Carlo Simulation

This chapter gives an overview of Monte Carlo simulation as a mathematical tool and as a complement or an alternative to real experiments.

8.1 Introduction and History

The Monte Carlo (MC) method is a general name of any method that uses a sequence of random numbers to perform different types of calculations. The method can be used to approximate solutions of problems in different areas. It was developed during the twenties century, but there are some older experiments from the second half of the nineteenth century which showed that different kind of deterministic problems can be solved by using random processes. A systematic development of the method started about 1944 and was named during the World War II, by a team working with the development of the atomic bomb. The method was named after the city of Monte Carlo, a centre of gambling. Because of the similarity between the games of chance played in the casinos, and the random numbers in a statistical simulation. The method was in the beginning used in order to solve nuclear physics problems, but soon it became a well known method that could be applied in different fields. For instance in mathematics, physics, economy, demography, etc. (Hammersley and Handscomb 1979, Marek et al. 2003)

8.2 Why a Simulation?

The objective of a simulation is to understand how something works in the reality. Simulations are experiments that are performed on designed models instead of real objects. In order to understand how something works and to prove the correctness, a model and its assumptions must be tested by repetitions of the behavior. Repetitions can be achieved by performing several physical experiments. But, in many cases the restrictions of both time and money makes it impossible to perform the experiments in the reality. A simulation is then a very competitive alternative to an experiment, since it is more cost and time effective in most cases. In some cases it might even be impossible to perform real experiments. It is possible to simulate how fast a mortal disease could be spread across the world and how large casualties that could be expected. Yet, no one would perform a real experimental study on a human population. A simulation is also a good tool to produce new artificial data from data collected by measurements, this technique is known as resampling. (Marek et al. 2003, Getachew 2003)

8.3 The Method

MC methods can handle two types of problems. They can either be probabilistic or deterministic. The difference between the two types is whether or not they are dealing with objects, operations or processes that involve randomness. A deterministic simulation deals with a process where all of the object and operations are non-random. In a probabilistic simulation, on the other hand, there are random objects or processes that can not be predicted. In the real world there are a lot of randomness, therefore it is hard to find a real situation that can be described as purely deterministic. For instance, a studied steel girder could have a
higher or lower strength, length, height, etc. compared to the values which the manufacturer
gives. Probabilistic simulations can deal with these variations.

There is only one requirement for probabilistic problems solved by MC simulations. They
must have solutions that can be described by probability density functions or probability mass
functions. (Hammersley and Handscomb 1979, Marek et al. 2003)

A mathematical model that describes the problem is created, and the input and output
variables are defined. The input values are randomly picked out of their probability
distribution functions, which are used to describe the input variables. One set of input values
will result in one set of output values, which will be stored in one way or another.

The law of large numbers is one of the fundamental things which the MC method is based on.
A short version of the law of large numbers is given by Marek et al. (2003)

“The empirical distribution of a random variable converges to the
theoretical one if the number of samples increases to infinity.”

The consequence of this law is that mathematical models are looped as many times as
necessary in order to get the desired accuracy of the results.

8.3.1 Central Limit Theorem

The central limit theorem (CLT) states why results from simulations can be very good
approximations of answers to some problems. The CLT states that the sum of \( n \) independent
random variables can be approximated by a normal distribution when \( n \) are large
(Hammersley and Handscomb 1979). This theorem is not general and there are some
limitations. The random variables are not allowed to vary in size too much. For instance, the
theorem is not valid if one of the variables is bigger than the sum of all other variables.
Another disadvantage of CLT is the “tail problem”. If \( n \) goes towards infinity then the
distribution function of the sum adopt the same shape as a normal distribution in the region
around the mean value. But, the values that are much lower or higher than the mean value
(values in the tail) do not adopt the normal distribution shape as quickly as the central regions
of the normal distribution.

8.3.2 Random Variables

A classification of random variables can be made in three basic types: attributes, counts and
measurements. Attributes are criteria which a sample either fulfills or not. The number of
samples that fulfill an attribute can be summarized in order to create counts. As an example,
the attribute could be “heavier weight than 70 kg”. If this attribute are tested on 1000 students,
the sample, one could create counts showing how the distribution is between students that
weigh more or less than 70 kg. Attributes and counts are both discrete variables.
Measurements are on the other hand continuous variables, which can be described by
numerical value. For instance length, weight, temperature, load etc. (Marek et al. 2003)

A random variable can be purely discrete, purely continuous or a combination of both discrete
and continuous. A purely discrete variable, which are defined at discrete values only, can be
represented by a Probability Mass Function (PMF) which shows how the variable is
Monte Carlo Simulation

8.3.3 Limitations

MC simulations based on experimental data are dependent on how good the experimental data are. The sample can be more or less suitable for the problem, and the result of the simulation can never be better than the input. If the input contains invalid data, then the output will contain invalid data as well. This is known as the axiom “garbage in garbage out”. Simulations will never give answers that are absolutely correct. The method can nevertheless be very useful in order to solve complicated problems. The question that has to be answered is, how likely it is that the answer is wrong and how much wrong might the answer be. If the uncertainties can be controlled then it will be possible to achieve an answer which can be treated as valid with a certain probability.

8.4 Structure of Monte Carlo Simulations

The structure of a MC simulation is almost the same, no matter what type of problem that is studied. Figure 8:2 is an illustration of the structure of a simulation, and describes how the different components and steps are connected. The following sections summarize how Marek et al. (2003) describe the major components and the basic structure of MC simulations.

8.4.1 Random Input Values

The probability distributions of the input values must be known in one way or another. One way of obtaining the probability distribution is to perform real experiments and record the data from the measurements that are done. If no tests can be performed and no historical data are available, it might be possible to create a probability distribution using theoretical knowledge. If there is little knowledge available about the problem that is studied, then it might be necessary to use some general theoretical distribution, like uniform, normal, triangular etc. The character of the problems governs which type of distribution that will be selected.

Figure 8:1 Illustration of distributions for discrete and continuous variables.

(a) PMF for a discrete variable. (b) PDF for a continuous variable

distributed. Rolling a dice would generate a PMF. Figure 8:1a shows a PMF for a discrete variable. The sum of the heights of the bars shall be equal to 1. Random variables that are purely continuous are described by continuous functions. These functions are called Probability Density Functions (PDF). Figure 8:1b shows a PDF for a continuous variable, the area under the curve shall equal 1.
8.4.2 Random Number Generation

To be able to perform a MC simulation there must be a source from which random numbers can be generated. The source is the probability distribution that is created in one way or another (see 8.4.1 Random Input Values). The generation process is normally made in two steps. The first step is to generate random numbers out of a uniform distribution. The second step is to convert the result from the first step into the specified probability distributions. The outcome of the second step is single values from the probability distributions of the input variables. These values will be used as input data in the system model, see Figure 8:2.

![Figure 8:2 Monte Carlo simulation scheme, after Marek et al. (2003).](image)

8.4.3 System Model

Problems taken from real life are often very complex and involve a large amount of variables. The real life situation must be translated into a mathematical model which describes the real situation as good as necessary. The creation of a system model involves a compromise between the complexity of the model and the accuracy. The model should also be designed in a way which makes it possible to perform time effective calculations. The studied problem must be described by at least one variable that has a probability density function or a probability mass function. If no probability function is involved in the model, then there will be no need of a MC Simulation.

8.4.4 Recording the Results

Results from simulations can be recorded and presented in different ways, depending on the type of problem and the aim of the simulation. In many cases there is no need of information from a specific calculation, a few statistical parameters are often the only values that are interesting.
Sometimes it can be useful to illustrate the results from the simulations by some sort of diagrams. Histograms are one way of presenting the results. Some important choices must be done when a histogram is created. The number of distribution categories and their intervals must be decided. Each simulation step will result in one value which will be sorted into one of the distribution categories, each category will be illustrated by a histogram bin. The choice of the number of histogram bins will affect the result and cause some errors if the graphical result is compared to the numerical result. If a continuous variable is involved in the calculation then it will always be a loss of information using histograms. The loss of information can be reduced by using more histogram bins.

The most time and space consuming way of recording the results is to record all output values and sometimes also all input values. If this is done, every single calculation can be reconstructed and studied. This type of recording can give enormous amounts of data, which has to be stored. Therefore, it is mostly used in cases where the solution of the problem is not as important as the understanding of the whole system.
PART 2

Mathematical Models, Simulations, and Calculations
9  Example Bridge - Leduån Bridge

The bridge that is used in the calculations, throughout this report, is a one span integral abutment bridge. It has a span length of 40 m, measured between the centres of the pile rows which are integrated in the abutment walls. It is a composite bridge with two I-girders made of steel, and a deck slab made of concrete C40/50. The bridge is a two lane roadway bridge, designed to be a crossing over the Leduån stream some kilometres west of the Swedish town Nordmaling. Some drawings of the bridge are shown in Figure 9:1 and 9:2.

![Figure 9:1 View of the bridge over the Leduån River.](image1)

![Figure 9:2 Cross-section view of the superstructure.](image2)
The bridge girders are produced in three sections. Two end girders with the same dimensions and a midspan girder. The bridge girder dimensions are shown in Figure 9:3 and Table 3:1.

![Figure 9:3 Drawing of the bridge girders, seen from above.](image)

### Table 9:1 Steel girder dimensions.

<table>
<thead>
<tr>
<th>End girders</th>
<th>dim [mm]</th>
<th>material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper flange</td>
<td>25x500</td>
<td>S460M</td>
</tr>
<tr>
<td>Web</td>
<td>13x1221</td>
<td>S355J2G3</td>
</tr>
<tr>
<td>Lower flange</td>
<td>36x800</td>
<td>S460M</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Midspan girders</th>
<th>dim [mm]</th>
<th>material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper flange</td>
<td>25x600</td>
<td>S460M</td>
</tr>
<tr>
<td>Web</td>
<td>11x1234</td>
<td>S355J2G3</td>
</tr>
<tr>
<td>Lower flange</td>
<td>45x800</td>
<td>S460M</td>
</tr>
</tbody>
</table>

Six end bearing piles are supporting each abutment. The piles used in the construction are steel pipe piles, RR170x10 see Table 9:2, which are driven in straight lines perpendicular to the longitudinal bridge axis. Pre-drilled holes with a depth of 2 m are performed at each pile position. Steel pipes with a diameter of 600 mm are installed, at the top 2 meters, as a shelter to the piles. Styrofoam plates are installed inside of these piles as shown in Figure 9:4. The pipes are then filled with loose sand that surrounds the piles.

![Figure 9:4 Cross-section of the upper part of the pile, surrounded by a steel pipe.](image)
Table 9:2 Pile dimensions and properties, RR170x10 from Ruukki.

<table>
<thead>
<tr>
<th>Without corrosion</th>
<th>2.4 mm corrosion</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>210 GPa</td>
</tr>
<tr>
<td>Ø</td>
<td>168.3 mm</td>
</tr>
<tr>
<td>t</td>
<td>10 mm</td>
</tr>
<tr>
<td>A</td>
<td>4973 mm²</td>
</tr>
<tr>
<td>W_p</td>
<td>250.9 cm³</td>
</tr>
<tr>
<td>I</td>
<td>1564 cm⁴</td>
</tr>
<tr>
<td>EI</td>
<td>3284 kN/m²</td>
</tr>
</tbody>
</table>

The bridge designer’s original idea was to use different type of piles in the two abutments. Therefore, an alternative pile cross-section was dimensioned. H-piles of type HEM120 were suggested to be used in one of the abutments. The number of piles and their positions were supposed to be the same as for the pipe piles. However, these piles could not be delivered in time and the pipe piles were used in both abutments. In this report, fatigue calculations are performed mainly for the pipe piles, but also for the alternative pile cross-section HEM120. This is done in order to find out if and how different cross-sections affect the fatigue lifetime. The dimensions and properties of the HEM120 cross-section are presented in Table 9:3.

Table 9:3 Alternative pile dimensions and properties, HEM120.

<table>
<thead>
<tr>
<th>Without corrosion</th>
<th>2.4 mm corrosion</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>210 GPa</td>
</tr>
<tr>
<td>h</td>
<td>140 Mm</td>
</tr>
<tr>
<td>b</td>
<td>126 Mm</td>
</tr>
<tr>
<td>t</td>
<td>21.0 mm</td>
</tr>
<tr>
<td>d</td>
<td>12.5 mm</td>
</tr>
<tr>
<td>R</td>
<td>12 mm</td>
</tr>
<tr>
<td>A</td>
<td>6641 mm²</td>
</tr>
<tr>
<td>I_x</td>
<td>2018 cm⁴</td>
</tr>
<tr>
<td>I_y</td>
<td>703 cm⁴</td>
</tr>
</tbody>
</table>
Figure 9:5 shows a view of the abutments and Figure 9:6 an illustration of how the piles are located in the abutments, seen from above.

**Figure 9:5** Abutment view drawing.

**Figure 9:6** Drawing of pile locations, seen from above.
10 Temperature Models

The temperature has been studied at five different locations in Sweden, see Figure 10:1. One of the locations, Karesuando, is chosen since it is the location in Sweden with the largest temperature differences during a year, according to Alexandersson et al. (1991). The four other locations were chosen to be able to study how the location of the bridge will affect the fatigue of the integral abutment piles. It is possible that there are other places in Sweden which have larger temperature differences than Karesuando, but temperature data from those places have not been accessible in this study. Temperature stations that are located in the mountain regions of Sweden, with no infrastructure in the surrounding area, have been excluded from the selection.

Figure 10:1 Map of Sweden, showing the locations where the temperature has been studied.

10.1 Seasonal Temperature Changes

Temperature measurements from different places indicate that seasonal temperature changes are varying almost like a cyclic sinus wave (Dicleli and Albhaiasi 2004, Arsoy 2000). Temperature measurements, in Stockholm, during a period of 5 years are used to illustrate the cyclic behaviour, see Figure 10:2. The input data are measured at the Old Astronomical Observatory Building in Stockholm, and compiled by Moberg et al. (2002, 2005).

Figure 10:2 Seasonal temperature changes in Stockholm during a period of five years.
10.1.1 Mathematical Model of Seasonal Temperature Changes

A mathematical model which describes how the daily mean temperature is varying, over a period of time, can be created by using a sinus curve that is adapted to measured temperatures at the specific location. The mean temperature of each month, at the five studied locations, can be obtained from *Temperature and Precipitation in Sweden 1961-90* (Alexandersson et al. 1991). These monthly mean temperatures are based on daily measurements from 1961-1990.

Seasonal temperature variations can be represented by a sinus curve with a period of one year. The amplitude of the curve can be adjusted to temperature measurements that are available. The sinus curve can also be shifted horizontally and vertically in order to fit the measured temperatures. By studying the variations in temperature over a historic period of several years, it might be possible to make a good estimation of the maximum and minimum daily average temperature during a year, $T_{s, \text{max}}$ and $T_{s, \text{min}}$. These values are used to calculate the amplitude of the sinus function, $T_{s, \text{amp}}$, and the mean temperature, $T_{s, \text{m}}$, of the seasonal average temperature curve, see (10.1) and (10.2).

$$T_{s, \text{amp}} = \frac{T_{s, \text{max}} - T_{s, \text{min}}}{2} \quad (10.1)$$

$$T_{s, \text{m}} = \frac{T_{s, \text{max}} + T_{s, \text{min}}}{2} \quad (10.2)$$

An expression for the seasonal variations in daily average temperature, $T_s$, can be written as

$$T_s = T_{s, \text{amp}} \cdot \sin\left(\frac{t_d}{365} \cdot 2\pi - t_0\right) + T_{s, \text{m}}, \quad (10.3)$$

where $t_d$ is the time measured in days, and $t_0$ is a factor which is introduced in order to shift the sinus curve horizontally to adjust it to measured temperature data. By changing this factor it is possible to adjust the model to different locations. In some cases the mathematical model seems to describe the real temperature better if also $T_{s, \text{amp}}$ and $T_{s, \text{m}}$ are adjusted slightly to fit the curve to the measured values. *Figure 10:3* shows an example of how the daily mean temperature is varying during a year in Kiruna. The solid line represents the mathematical sinus model, and the dots are representing the monthly mean temperature during the period from 1961-1990. Input data to *Figure 10:3* are presented in *Appendix G*.

![Mathematical mean temperature model for Kiruna, compared with historical data.](image)
The mathematical model is focused on the extreme values among the monthly mean temperatures during a year. In this case with Kiruna it differs up to almost 1.7 degrees, in April, between the measured temperature and the model. Yet, the model fits the historical data quite good. The difference is less than 0.6°C in eight of twelve months, and the mean difference is less than 0.7°C.

There are probably other functions that can describe the seasonal temperature changes more exactly than this simple model. The question is if it is worth the effort to construct a more complex temperature model, when it is obvious that temperatures never can be predicted exactly. The purpose of this model is to generate temperatures which primary shall be used to calculate thermal bridge movements, and secondary to calculate the amplitude of the strain cycles that occur in the abutment piles. The seasonal temperature variations are interesting mostly in order to describe how large the seasonal strain cycles will be. The extreme values will be most important. Daily temperature changes will also take place and induce movements and strains in the piles, on top of the seasonal contribution. These daily changes and the strain amplitudes that they generate will be more interesting, than the exactly temperature that they are varying around, especially in the months with daily mean temperature far away from the extreme values. Therefore, the seasonal temperature model should be focused on covering the extreme values, and the daily temperature model should be focused on amplitudes rather than mean values.

The temperature data have mainly been handled in Microsoft Excel, and since this program can produce regression lines this function has been tested and compared to the sinus model. Excel can however only produce polynomial regression lines up to sixth degree. Figure 10:4 illustrates results from the sinus model and the polynomial regression line produced by Excel. The equations of the regression line and the sinus curve are also shown in the figure.

The polynomial regression line, of sixth degree, is obviously not that good to predict the extreme temperature in July. The sinus curve on the other hand, has been adjusted to fit the extreme values relatively good, but is not as good as the polynomial regression line to predict the temperature in the month between the extreme values. A polynomial regression line of a higher degree might describe the temperature in a better way.
The sinus model will be used in the temperature calculations in this report. Mostly because of the facts that it is cyclic, easy to handle and can easily be adjusted to the extreme values. The extreme values are assumed to give rise to the largest contribution to low-cycle fatigue in integral abutment piles, from seasonal temperature variations.

10.2 Daily Temperature Changes

Daily temperature variations can be modelled in many different ways. For instance by sinus functions with periods of one day and with amplitudes that is adjusted to measured values, or by statistical probability distributions which gives the highest and lowest temperature during a day. The maximum and minimum temperatures could be assumed to take place in 24 hour intervals, and with 12 hours between the maximum and minimum values. The temperatures between the daily extreme values could easily be described by connecting the maximum and minimum values with straight lines or some trigonometric function.

If there are no measurements of daily temperatures changes accessible for the specific location, then it might be necessary to use a simplified temperature model. A simplified mathematical model could be based on results from field studies made at other locations. Observations from several field tests are indicating that daily temperature induced movements have larger magnitudes in winter than in summer (Arsoy 2000). A mathematical model for daily temperature changes should be able to take these variations into consideration. Three different mathematical models are proposed in order to describe daily temperature variations.

10.2.1 Model 1

This is the most simplified and most conservative temperature model. Instead of using daily temperature changes, monthly variations in temperature are studied. If the maximum and minimum values of daily temperatures are studied over a whole month, instead of a day, then it would generate a higher temperature range than a single day, and the model would be conservative. The benefit of studying temperature variations over a whole month, instead of a day, is the reduced need for input data. Maximum and minimum temperatures for each month are in general easy to get access to. Since meteorological institutes all over the world are collecting these data in order to create comparable climate information, according to the general directions from The World Meteorological Organization (SMHI 2006).

Arsoy (2000) used a model based on monthly temperature variations in his study, because of the reasons that it was easier to obtain the monthly temperature data, the reduced amount of input data was easier to handle, and the answer would still be on the safe side.

The monthly maximum and minimum temperatures, $T_{m,\text{max}}$ and $T_{m,\text{min}}$, that are used in the model are mean values of the maximum and minimum daily temperatures during a month. Equation (10.4) can be used to describe the daily variations in temperature, according to Model 1.
\[ T_d = \frac{\Delta T_{m,i} \cdot \cos(t_d \cdot 2\pi)}{2} \]  
\[ \Delta T_{m,i} = T_{m,i,\text{max}} - T_{m,i,\text{min}} \]

\( i = \text{January, February, ….. , December} \)

The expression of the daily temperature variations will change from one month to another. Index \( i \), which represent the months, is therefore introduced in the equations. Figure 10:5 illustrates an example of a temperature distribution during a year according to Model 1. The daily temperature changes have been superposed on the seasonal changes. The period of the daily changes has been modified in Figure 10:5, it has been plotted as three times longer than in the reality. This is only done in order to get a clearer and more illustrative figure.

![Figure 10:5 Example of a result from temperature Model 1](image)

<table>
<thead>
<tr>
<th>Input values</th>
<th>( T_{m, \text{max}} ) [(^\circ\text{C})]</th>
<th>( T_{m, \text{min}} ) [(^\circ\text{C})]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan</td>
<td>-2</td>
<td>-28</td>
</tr>
<tr>
<td>Feb</td>
<td>-3</td>
<td>-26</td>
</tr>
<tr>
<td>Mars</td>
<td>2</td>
<td>-18</td>
</tr>
<tr>
<td>April</td>
<td>6</td>
<td>-10</td>
</tr>
<tr>
<td>May</td>
<td>12</td>
<td>-2</td>
</tr>
<tr>
<td>June</td>
<td>19</td>
<td>7</td>
</tr>
<tr>
<td>July</td>
<td>21</td>
<td>9</td>
</tr>
<tr>
<td>Aug</td>
<td>16</td>
<td>3</td>
</tr>
<tr>
<td>Sept</td>
<td>14</td>
<td>-1</td>
</tr>
<tr>
<td>Oct</td>
<td>8</td>
<td>-10</td>
</tr>
<tr>
<td>Nov</td>
<td>3</td>
<td>-18</td>
</tr>
<tr>
<td>Dec</td>
<td>1</td>
<td>-22</td>
</tr>
</tbody>
</table>

This temperature model does not describe the temperature variations in a realistic way, the real situation are much more random. The daily changes in temperature would certainly be overestimated over a year, by this model, but it does not take into account the probability of even higher temperature changes at single days. This model has its benefit in the simplicity and could be used as a first trial, and to evaluate if the results are acceptable in spite of the fact that the model is conservative. If the purpose is to simulate the reality as good as possible, another model should be used.

### 10.2.2 Model 2

This model is based on probability distributions in one way or another. The simplest way of simulating daily temperature variations would be by using a probability distribution with a mean value, \( \mu \), that equals zero. Random numbers could then be generated from this probability distribution and superposed on top of the seasonal mean temperature curve, see Figure 10:6.
Figure 10:6 Illustration of how daily temperatures could be added to the seasonal temperature curve by using different probability distributions.

The daily maximum and minimum temperatures could be generated from a uniform temperature distribution between the daily maximum and minimum temperatures, which can be obtained from temperature databases at SMHI. This model will indeed not take into consideration the extreme temperatures that do not occur too often. These extreme values might however be very important in the analysis of low-cycle fatigue in integral abutment piles. Therefore, a normal distribution is suggested instead, in order to take the “tail” temperatures into consideration. The normal distribution could be adapted to the measured extreme values that statistically occurs one time in fifty years, or some other time interval.

The big problem with a normal distribution, which is adapted to extreme values that occur in certain time intervals, is that the probability for the maximum and minimum temperatures are highest around the daily mean temperature. The model does not seem to agree with daily measured maximum and minimum temperatures available in *Sveriges Nationalatlas – Klimat, sjöar och vattendrag* (1995). A better agreement would be achieved if the maximum and minimum daily temperatures were modelled by two separate normal distributions. The mean values of the distributions could be set as the monthly mean values of the daily maximum and minimum temperatures. The standard deviations could be adapted to the characteristic temperatures that statistically occur with a certain probability. The mean values of the daily maximum and minimum temperature during a month are often quite easy to get access to. *Klimatdata för Sverige* (Teasler 1972) has in this study been used as the source for daily maximum and minimum values. These values are unfortunately relatively old, from 1931-1960. The current standard normal period is 1961-1990, but temperatures from this period has only been available in the form of isothermal maps, in this study. The data from 1931-1960 has been compared with the isothermal maps from 1961-1990, and they seem to agree quite well with each other. It might differ up to one degree some month. The overall changes in temperatures should not affect the fatigue analysis, while the daily temperature changes are modelled with a safety margin that surely is larger than one degree.
Annual variations in daily maximum and minimum temperature can be modelled in the same way as the seasonal mean temperature, and represented by sinus functions. The daily changes are modelled by a normal distribution which is added on top of the sinus functions. Figure 10:7 is illustrating a simulation of the daily maximum and minimum temperature during a year. The simulation has been performed with input data for Karesuando, see Appendix G. The normal probability distribution and the random generation of numbers, has been performed with the computer programs Anthill Lite and Microsoft Excel with the Analysis ToolPak.

The normal distribution of the daily temperature changes must somehow be established in real measurements. The Swedish Bridge Code - BRO2004 contains two isothermal maps over Sweden in which the maximum and minimum characteristic temperatures are presented, see Appendix E. These temperatures occur in intervals of 50 years. The Swedish National Board of Housing, Building and Planning (Boverket) provides supplementary tables of max and min temperature at about 250 locations in Sweden. The maps and the tables are used in order to fit the temperature model to the probability of 0.02 (once in 50 years) that the given characteristic temperatures will occur once in a year. The adjustments and calculations of the normal distributions are presented in Chapter 11.

Information about the normal distribution, or Gaussian distribution, as a probability tool can be found in handbooks about statistics or probability. The theory behind the model will not be described in this report, only the equation and its parameters will be presented briefly. The probability distribution function are illustrated in Figure 10:8, along with the equation where \( \sigma \) are the standard deviation and \( \mu \) the mean value. The normal distribution will be denoted as \( N(\mu, \sigma) \) when it is used in the calculations.

\[
f(x) = \frac{1}{\sigma \sqrt{2\pi}} e^{-\frac{(x-\mu)^2}{2\sigma^2}}
\]
10.2.3 Statistical Model

If extensive temperature measurements have been done, every day during many years, it would be possible to create a probability distribution function for each time of a day when the temperatures have been registered. The temperature model should in some way also take the temperatures from the former hours or days into consideration. It would give the probability for the occurrence of a certain temperature at a certain time at a certain date. A source to such measurements could be the national meteorological institute. They have often measurements from at least 30 years available. In many cases, it would probably be possible to get all the temperatures that are needed. But the question is, if the benefits from using real temperature statistic are worth the effort of creating the database of probability functions that are needed.

It would indeed take some time to create all the probability functions that are needed, and to make a model which also takes into consideration the temperature during the former hours and days. Such a model would certainly describe the temperature changes in the most proper way statistically, among the suggested models. Yet, it would still only give an estimation of how the temperature changes could be, since there are no absolute answers.

10.3 Temperature Gradient

Temperature gradients in the superstructure will result in bending moments which will rotate the abutment and induce an additional bending moment in the piles. Temperature gradients will be time and weather dependent and give positive as well as negative bending moments. Positive moments occur when the deck is warmer than the girders, and negative moments occur when the girders are warmer than the bridge deck. Positive temperature gradients are expected to be largest in days with high solar radiations, while the negative temperature gradients are expected to be highest during nights when the ground surface are warmer than the air. Table 10:1 shows the characteristic values of positive and negative temperature gradients for a composite bridge, given by the proposal to Eurocode and BRO2004.

<table>
<thead>
<tr>
<th></th>
<th>$\Delta T^+$ [°C]</th>
<th>$\Delta T^-$ [°C]</th>
</tr>
</thead>
<tbody>
<tr>
<td>ENV 1991-2-5</td>
<td>15</td>
<td>-18</td>
</tr>
<tr>
<td>BRO2004</td>
<td>10</td>
<td>-5</td>
</tr>
</tbody>
</table>

Since thermal gradients will give moments that are varying from one time to another, they should be taken into account when a low-cycle fatigue analysis is made. A simple and conservative approach would be preferable, in order to keep the amount of input data rather low. No other values or models have been available than the design values from the codes. A conservative approach could therefore be to assume that the thermal gradient is varying between the negative characteristic value in the day and the positive characteristic value in the night. The highest and lowest values would be assumed to occur at the same time as the maximum and minimum daily temperatures. The advantage of this model would be the simplicity, and the disadvantage is that it would be a very conservative assumption, since the characteristic values given in the codes are based on temperature gradients that occur once in 50 years. The pile strains caused by temperature gradients should be overstated a lot, and another model is preferable.
A simulation of the positive and negative temperature differences from a probability distribution could also be an alternative. The probability distributions could be adapted to the characteristic temperature differences and the reduced temperature differences, given in Table 5:1 respectively Table 5:2. The simulation process is described in Chapter 11.2.

### 10.4 Effective Bridge Temperature Models

The Effective Bridge Temperature (EBT) has been presented in Chapter 5, and two different ways of transforming the shade air temperature into EBT have been discussed.

#### 10.4.1 EBT – According to Oesterle and Volz

The EBT model presented by Oesterle and Volz (2005) uses two temperature variables to calculate the maximum and minimum EBT. The variables are, the shade air temperature ($T_{\text{shade}}$) and the uniform temperature change due to direct solar radiation ($\Delta T_{\text{solar}}$). The shade air temperature at a specific location can be modelled from earlier measurements, according to the models in the previous sections. The variable that takes the effect from the solar radiation into consideration is not as easy to calculate. Oesterle and Volz are suggesting that the Solar Incremental Temperature, $T_1$, is used to calculate the effect from the solar radiation.

A source of the Solar Incremental Temperatures in Sweden has not been found. For that reason, this model cannot be used in the way it is supposed to be used.

#### 10.4.2 EBT – According to Eurocode

The EBT model given in the proposal to Eurocode, ENV1991-2-5, is only using the shade air temperature to calculate the EBT. The studied bridge is a composite bridge and classed as a Group 2 bridge, according to Eurocode. The relationship between the max/min shade air temperature and max/min EBT is given in Figure 5:4. The EBT for bridges in Group 2 can be calculated according to the following expression, which has been derived from Figure 5:4.

\[
T_{\text{eff max/min}} = 4 + 0.98 \cdot T_{\text{shade max/min}}
\]  

(10.6)

In this study, simulated daily maximum and minimum shade air temperatures are transformed into Effective Bridge Temperatures by equation (10.6). The EBTs are later on used to calculate the thermal movements of the abutments.
10.5 Comments

Model 1 could be a starting point if the temperature behaviour shall be analysed, since it is rather simple and conservative. That kind of temperature model has been used in laboratory tests to simulate thermal movements in integral abutment piles (Arsoy 2000). Model 1 is probably best suited for laboratory testing, since it reduces the need of input data to control the loading and unloading device. But, the temperature is obviously not varying in that perfect pattern in the reality. The model has some “tail problem” as well, while it does not take into consideration the extreme temperatures that might occur now and then. Therefore, I suggest that Model 2, with normal distributed maximum and minimum daily temperatures, is used in an analysis of low-cycle fatigue in integral abutment piles. The statistical model would certainly give a temperature distribution which statistically would be most correct, but I do not believe that it is worth the time and effort to construct such a model. No matter how much temperature measurements and information that is available, you can not predict the temperature.

Fields studies on integral abutment piles have showed that, daily temperature changes give strains that are approximately 20-40 % of the strain amplitude given by seasonal variations in temperature (Dicleli and Albhaisi 2004). Another observation from several field tests is that the daily temperature induced movements have a larger magnitude in the winter than in the summer (Arsoy 2000). If there is a shortage or a lack of daily temperature data and the only data available is the seasonal trend. Then it might be necessary to simplify Model 2, and use the observations mentioned above instead, to produce a mathematical model of daily temperature changes.
11 Temperature Simulations

Monte Carlo simulations, based on the models described in previous chapters, have been performed to simulate the daily temperature changes and the varying temperature differences between deck and girders.

11.1 Daily Temperatures

The daily temperature simulations that have been performed are based on temperature Model 2, presented in a previous chapter. The daily temperature variations are simulated by using normal distributions which are added on top of the seasonal trend for the daily maximum respectively minimum temperature. The standard deviations in the normal distributions are adjusted to the extreme values that occur statistically once in 50 years, see Table 11:1. The temperatures given in Table 11:1 can be found in the document BFS 2005:9 EBS2 published by the Swedish National Board of Housing, Building and Planning (Boverket 2005).

Table 11:1 Characteristic temperatures, which statistically occur one time in 50 years.

<table>
<thead>
<tr>
<th>Location</th>
<th>Maximum temperature [°C]</th>
<th>Minimum temperature [°C]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Karesuando</td>
<td>31</td>
<td>-48</td>
</tr>
<tr>
<td>Kiruna</td>
<td>30</td>
<td>-45</td>
</tr>
<tr>
<td>Umeå</td>
<td>29</td>
<td>-38</td>
</tr>
<tr>
<td>Stockholm</td>
<td>36</td>
<td>-29</td>
</tr>
<tr>
<td>Malmö</td>
<td>33</td>
<td>-22</td>
</tr>
</tbody>
</table>

One maximum and one minimum temperature are simulated every single day, that gives 18 250 daily max and min temperatures during a time period of 50 years. The statistical probability that a certain daily temperature shall exceed the characteristic temperatures is 1/18250.

The daily maximum and minimum temperature are modelled by sinus functions and normal distributions according to following equation,

$$T_{d,\text{max/min}} = T_{\text{max/min amp}} \cdot \sin \left( \frac{t_d}{365} \cdot 2\pi - t_0 \right) + T_{\text{max/min mean}} + N(\mu, \sigma_{\text{max/min}}). \quad (11.1)$$

The parameters in the sinus function have been established for five different locations, see Appendix G. The only parameter that can be used to fit the daily temperature variations to the characteristic temperatures is the standard deviation. Appropriate values for standard deviations for max and min temperatures at different locations are calculated by using a computer program, Anthill Lite. The number of simulation steps are limited by the program to 50 000 steps.

The calculations are performed iteratively. A value of the standard deviation is chosen and the temperature model are then simulating the daily temperature during 100 years, 36 500 max respectively min values are calculated. The results from the simulations are then compared to
the max and min temperatures that statistically are exceeded two times in 100 years. The probability that one of the daily max/min temperatures are exceeding the 50 year extreme value is $5.48 \times 10^{-5}$. The model are ran 10 times at each chosen standard deviation, and the temperature that is exceeded with a probability of $5.48 \times 10^{-5}$ is registered. The mean values of the results, from running the model 10 times, are then compared to the values in Table 11:1. The procedure continuous until the difference, between the values given in Table 11:1 and the simulated values, is less than 0.5 degree.

Figure 11:1 illustrates the result from a run of the model performed in order to find the standard deviation making the temperature exceed 30°C with a probability of once in 50 years.

MC simulations give the standard deviations shown in Table 11:2, for daily maximum and minimum temperature models at different locations. Several studies have showed that daily variations in temperature are larger in the winter than in the summer. The standard deviations for the minimum daily temperatures have therefore been separated into two standard deviations. One standard deviation for the six warmest months, and one for the six coldest. If only one standard deviation is used to describe the minimum temperature variations, based on the probability that a certain minimum temperature occurs once in fifty years. Then as a result of the rather high value of the standard deviation, the simulation of daily minimum temperature in summer could generate temperatures that are even higher than the maximum temperatures that occur with a probability of once in fifty years. The two different standard deviations are introduced to avoid these problems. Another kind of distribution could also be used as an alternative to avoid these problems.

<table>
<thead>
<tr>
<th>Standard deviations - $\sigma$ [°C]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Max temp</strong></td>
</tr>
<tr>
<td>Jan - Dec</td>
</tr>
<tr>
<td>Karesuando</td>
</tr>
<tr>
<td>Kiruna</td>
</tr>
<tr>
<td>Umeå</td>
</tr>
<tr>
<td>Stockholm</td>
</tr>
<tr>
<td>Malmö</td>
</tr>
</tbody>
</table>
The daily maximum and minimum shade air temperatures are simulated during the service lifetime of the bridge, 120 years. A graphical example of a result from a simulation of fifty years temperature variations in Kiruna, is given in Appendix I - Figure II. The temperatures generated by the simulations are converted into effective bridge temperatures, by using (10.6) from the Eurocode EBT-model. The simulated shade air temperatures, during 10 years in Kiruna, are compared with the calculated EBTs in Figure I2, Appendix I.

The difference between the simulated daily max and min temperature are summarized, and the mean difference is calculated. This value is compared to the mean difference between the daily max and min temperature given in Klimatdata för Sverige (Teasler 1972), see Table 11:3.

Table 11:3 Comparisons between simulated and statistical mean values of daily temperature variations [°C].

<table>
<thead>
<tr>
<th></th>
<th>Simulated</th>
<th>Statistical</th>
</tr>
</thead>
<tbody>
<tr>
<td>Karesuando</td>
<td>10.1</td>
<td>8.8</td>
</tr>
<tr>
<td>Kiruna</td>
<td>9.8</td>
<td>8.4</td>
</tr>
<tr>
<td>Umeå</td>
<td>9.4</td>
<td>8.3</td>
</tr>
<tr>
<td>Stockholm</td>
<td>7.7</td>
<td>5.9</td>
</tr>
<tr>
<td>Malmö</td>
<td>7.6</td>
<td>6.9</td>
</tr>
</tbody>
</table>

The mean value of the simulated daily temperature variations is larger than the mean value of the statistical temperature variations, at all locations. The difference is between 10 and 17% for all locations except Stockholm that has a difference of 30%. Therefore, the daily temperature model will be conservative. The safety margin against the statistical values could be minimised, but my opinion is that it is better too keep it a bit on the safe side in order to study a case that are at least as tough as the reality. A safety margin in daily temperature variations could also be justified by the fact that the Eurocode EBT model is rather simplified and dependent only on shade air temperatures. The safety margin can be seen as a way to deal with unexpected variations in the other weather factors, which the EBT model does not take into consideration.

11.2 Temperature Differences Between Deck and Girders

The MC simulation program Anthill Lite was used to adapt probability distributions to the temperature differences that occur with a certain probability, according to ENV 1991-2-5. One negative and one positive temperature difference is simulated every day during the bridge service lifetime. They are assumed to take place at the same time as the coldest respectively warmest daily shade air temperatures.

The probability distributions are mainly adapted to the more extreme temperature differences that occur once in a year and once in 50 years, since these are assumed to give the largest contribution to the fatigue. The quasi permanent temperature differences are adapted to the mean value as good as possible without affecting the extreme values to much. A log-normal distribution was chosen since it seemed to be most suitable. A normal distribution could be an alternative but if it is adapted to the extreme values it would overstate the more frequently occurring values. Figure II:2 illustrates how the negative and positive temperature differences are described by two suitable log-normal distributions. The temperature differences that occur with a certain probability according to ENV 1991-2-5 are compared to the temperatures in the log-normal distribution that occur with the same probability, see Table 11:4.
Temperature Simulations

Low-cycle Fatigue of Steel Piles in Integral Abutment Bridges

Figure 11:2 Log-normal distributions used to simulate negative respectively positive temperature differences.

Table 11:4 Simulated temperature differences compared with temperature differences from ENV1991-2-5.

<table>
<thead>
<tr>
<th>Probability</th>
<th>$\Delta T^-$ [°C]</th>
<th>$\Delta T^+$ [°C]</th>
</tr>
</thead>
<tbody>
<tr>
<td>ENV 1991</td>
<td>Sim</td>
<td>ENV 1991</td>
</tr>
<tr>
<td>once in 50 years</td>
<td>-180</td>
<td>-17.9</td>
</tr>
<tr>
<td>once in 1 year</td>
<td>-14.4</td>
<td>-14.4</td>
</tr>
<tr>
<td>once in 14 days</td>
<td>-10.8</td>
<td>-11.3</td>
</tr>
<tr>
<td>Quasi-permanent value</td>
<td>-9.0</td>
<td>-8.7</td>
</tr>
</tbody>
</table>

The vertical temperature gradient in the superstructure is assumed to vary linearly between the negative and positive temperature differences that occur each day. Figure 11:3 illustrates a result from a simulation of varying temperature gradients during four months.

Figure 11:3 Graphical result of a simulation of varying temperature gradient during four months.
11.3 Thermal Movements

The expansion and contraction of the superstructure, due to changes in EBT, can be described as following,

$$\Delta L = \alpha \Delta T_N L_b.$$  \hspace{1cm} \text{(11.2)}

The length of the superstructure at the studied bridge, $L_b$, is 40.0 m and the thermal coefficient, $\alpha = 1.0 \cdot 10^{-5} \text{°C}^{-1}$, is taken from the Swedish Bridge Code - BRO2004. It is valid for both steel and concrete. The changes in EBT, $\Delta T_N$, are taken from the effective bridge temperatures that have been calculated from the simulated shade air temperatures. The variations in the length of the bridge can be plotted as a function of time, see Figure 11:4. The bridge deck and the top of the abutment walls are assumed to be cast the 1st of June in a shade air temperature of 10°C. After that date, the joints between the superstructure and the abutment walls are treated as rigid.

![Figure 11:4 Varying bridge length, during 3 years, of a composite bridge with a single span of 40.0 m, located in Kiruna.](image)

Thermal movements can either be assumed to cause displacements of the same size at either abutments, or displacements that are unevenly distributed between the abutments. In this study, the displacements are assumed to be the same at both abutments.
Temperature Simulations
12 Soil Calculations

Investigations of soil conditions below the abutments have been performed. Some of the results are presented in Table 12:1 and Figure 12:1. These parameters are taken from the design drawings of the Leduån Bridge.

<table>
<thead>
<tr>
<th>Level</th>
<th>Soil type</th>
<th>Soil parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.000 m – -2.000 m</td>
<td>Fine sand and fine sandy silt</td>
<td>$\phi_k = 28-30^\circ$</td>
</tr>
<tr>
<td>-2.000 m – -6.000 m</td>
<td>Fine sandy silt and silty fine sand</td>
<td>$\phi_k = 30^\circ$</td>
</tr>
<tr>
<td>-6.000 m – -20.000 m</td>
<td>Silt / Clay</td>
<td>$c_{uk} = 40$ kPa</td>
</tr>
</tbody>
</table>

Figure 12:1 Illustration of soil conditions below the abutments.

12.1 Lateral Soil Stiffness Parameter – $k_h$

The bottoms of the abutments are situated at level +1.090 m, and the piles are driven in pre-drilled holes with a depth of 2 m. The pre-drilled holes are sheltered by steel pipes, $\varnothing 600$ mm, which are filled with loose sand and styrofoam plates. A properly filled pre-drilled hole shall be treated as soil without any lateral soil stiffness (Abendroth and Greimann 2005). The top 0.6 m of the piles will be surrounded by crushed stone, as erosion protection. The piles will be sheltered by the back wall, and the lateral soil stiffness from the erosion protection is neglected. The undisturbed soil starts at level –0.910 m. The lateral soil stiffness is calculated from this level and down to a depth of half of the critical length of a pile, $l_c$, since this length will be the active part of the pile according to the model developed by Abendroth and Greimann (1989, 2005). A new coordinate axis, $z$, is introduced in order to make the calculations easier. It has its zero point at the bottom of the abutment (+1.090) and is positive downwards. The lateral soil stiffness, $k_h = k_k \cdot d$, for the different layers are calculated according to Appendix 3-4 in BRO2004. The lateral soil stiffnesses are mean values for movements up to the limit pressure, $q_k$, above this level the soil stiffness is constant.
Granular soil – long and short term loading

\[ k_k = \frac{n_h \cdot z}{d} \]  
(12.1)

- \( n_h \) coefficient of subgrade reaction  
- \( z \) soil depth  
- \( d \) pile width

Cohesive soil

Long term loading

\[ k_k = \frac{50 \cdot c_{u_k}}{d} \]  
(12.2)

\[ q_k = 6 \cdot c_{u_k} \]  
(12.3)

Short term loading

\[ k_k = \frac{200 \cdot c_{u_k}}{d} \]  
(12.4)

\[ q_k = 9 \cdot c_{u_k} \]  
(12.5)

- \( c_{u_k} \) undrained shear strength

The groundwater level is situated at ±0.000 m. All of the undisturbed granular soil layers are below the water table. According to geotechnical data, the relative soil stiffness is “very low”. This gives the following coefficients of subgrade reaction.

- \( n_h = 1.5 \text{ MN/m}^3 \)  
  (very low relative soil stiffness, below groundwater level)
- \( n_{hm} = 4.5 \text{ MN/m}^3 \)  
  (medium relative soil stiffness, below groundwater level)

The subgrade reaction modulus \( n_{hm} \) is used to proportion the \( k_k \cdot d \) values to different soil stiffnesses, since \( k_k \cdot d \) values are given only for medium relative soil stiffness in BRO2004. The following maximum values of \( k_k \cdot d \) are allowed above the groundwater level, according to BRO2004. Below the groundwater level the given maximum values are reduced to 60%.

Sand  \( (k_k \cdot d)_{\text{max}} = 12 \text{ MN/m}^3 \)
Silt  \( (k_k \cdot d)_{\text{max}} = 6 \text{ MN/m}^3 \)

\[ \text{Sand (above GWL)} \quad z = 0 - 1.090 \]
\[ (k_k \cdot d)_{\text{max}} = \left( k_k \cdot d \right)_{\text{max}} \frac{n_{hl}}{n_{hm}} = 12 \cdot \frac{2.5}{7.0} = 4.286 \text{ MN/m}^3 \]
\[ (k_{kl} \cdot d) = n_{hl} \cdot z_{\text{max}} = 2.5 \cdot 1.090 = 2.725 \text{ MN/m}^3 \]

\[ \text{Sand (below GWL)} \quad z = 1.090 - 3.090 \]
\[ (k_k \cdot d)_{\text{max}} = 0.6 \cdot (k_k \cdot d)_{\text{max}} \frac{n_{hl}}{n_{hm}} = 0.6 \cdot 12 \cdot \frac{1.5}{4.5} = 2.4 \text{ MN/m}^3 \]
\[ (k_{kl} \cdot d) = n_{hl} \cdot z_{\text{max}} = 2.5 \cdot 3.090 = 7.725 \text{ MN/m}^3 \]
Soil Calculations

\[
\text{Silt} \quad z = 3.090 - 7.090
\]
\[
(k_{k2}d)_{\text{max}} = 0.6 \cdot (k_k \cdot d)_{\text{max}} \quad \frac{n_{h2}}{n_{hn}} = 0.6 \cdot \frac{1.5}{4.5} = 1.2 \text{ MN/m}^3
\]
\[
(k_{k2}d) = n_{h2}z_{\text{max}} = 1.5 \cdot 7.090 = 10.635 \text{ MN/m}^3
\]

\[
\text{Clay/Silt} \quad z = 7.090 - 21.090
\]

As a first assumption, this layer was assumed to be below the part of the piles that are active, \(l/2\). The iterative calculation process, described in next section, verified later that this was a correct assumption. Therefore, no calculations of the soil stiffness in the Silt/Clay layer have been done.

### 12.2 Equivalent Uniform Lateral Soil Stiffness Parameter - \(k_{eh}\)

The varying soil stiffness parameter, \(k_h\), is transformed into an equivalent uniform lateral soil stiffness parameter, \(k_{eh}\). The calculations are done by an iterative process described in detail in Appendix B. The following data were used as input to the calculations, together with the soil parameters calculated in the previous section.

**No corrosion** | **2.4 mm corrosion**
---|---
\(E = 210 \text{ GPa}\) | \(E = 210 \text{ GPa}\)
\(I = 1564 \text{ cm}^4\) | \(I = 1133.6 \text{ cm}^4\)

\[\downarrow \quad \text{Iterative calculations} \quad \downarrow\]

\[k_{eh} = 2.246 \text{ MN/m}^2\]
\[l_{c/2} = 2.199 \text{ m}\]

\[k_{eh} = 2.283 \text{ MN/m}^2\]
\[l_{c/2} = 2.021 \text{ m}\]

The left part of Figure 12:2 illustrates the varying lateral soil stiffness parameter, \(k_h\), down to the silt/clay layer at a depth of 7.090 m. The right part illustrates the \(k_{eh}\) parameter and the active length for a pile with no corrosion and one that has corroded to a depth of 2.4 mm.

---

**Figure 12:2** Diagrams over the varying soil stiffness parameter (\(k_h\)) and the equivalent uniform soil stiffness parameter (\(k_{eh}\)).
Alternative soil model
According to Abendroth and Greimann (2005), properly filled pre-drilled holes shall be treated as soil without any lateral stiffness in the calculation of the equivalent cantilever length. In this report, an alternative soil model has also been used to calculate the equivalent cantilever length. This model takes into consideration the lateral stiffness of the sand in the predrilled holes. The calculations are done in order to make a comparison between the results. The results of these calculations are shown in Figure 12:3.

![Diagram](Image)

**Figure 12:3** Diagrams over the varying soil stiffness parameter ($k_h$) and the equivalent uniform soil stiffness parameter ($k_{eh}$), for the alternative soil model.

Alternative pile cross-section
The varying soil stiffness parameter, $k_h$, for the HEM120 piles is transformed into $k_{eh}$ in the same way as the calculations shown above for the pipe piles. The results from the calculations are presented in Table 12:2 and Table 12:3.

<table>
<thead>
<tr>
<th>Table 12:2 Results from calculations of $k_{eh}$, original soil model.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Original soil model</strong></td>
</tr>
<tr>
<td><strong>No corrosion</strong></td>
</tr>
<tr>
<td>$l_y$ 703 cm$^4$</td>
</tr>
<tr>
<td>$k_{eh}$ 2.329 MN/m$^2$</td>
</tr>
<tr>
<td>$l_c/2$ 3.569 m</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 12:3 Results from calculations of $k_{eh}$, alternative soil model.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Alternative soil model</strong></td>
</tr>
<tr>
<td><strong>No corrosion</strong></td>
</tr>
<tr>
<td>$l_y$ 703 cm$^4$</td>
</tr>
<tr>
<td>$k_{eh}$ 1.168 MN/m$^2$</td>
</tr>
<tr>
<td>$l_c/2$ 2.121 m</td>
</tr>
</tbody>
</table>
12.3 Calculations of Equivalent Cantilever Length

Calculations are made according to the concept described in Appendix B.

**Equivalent cantilever length – \( L_{equ} \)**

\[
L_{equ} = l_c + l_u
\]  

(12.6)

- \( l_u = 2\,600\,\text{mm} \) (depth of pre-drilled hole + not embedded length)

**Critical pile length without corrosion – \( l_c \)**

\[
l_c = 4 \cdot 4 \sqrt[3]{\frac{E_p I_p}{k_{eh}}} ,
\]  

(12.7)

- \( E_p = 210\,\text{GPa} \)
- \( I_p = 1564\,\text{cm}^4 \)
- \( k_{eh} = 2.246\,\text{MN/m}^2 \)
- \( l_c = 4.415\,\text{m} \)

**Critical pile length with 2.4 mm corrosion allowances – \( l_c \)**

\[
l_c = 4 \cdot 4 \sqrt[3]{\frac{E_p I_p}{k_{eh}}} ,
\]  

(12.8)

- \( E_p = 210\,\text{GPa} \)
- \( I_p = 1133.6\,\text{cm}^4 \)
- \( k_{eh} = 2.283\,\text{MN/m}^2 \)
- \( l_c = 4.247\,\text{m} \)

**Embedded length of \( L_{equ} \) – \( l_c \)**

*Figure 12.4* is re-created from a diagram developed by Abendroth and Greimann (1989, 2005). The original functions have not been found, and polynomial regression lines of sixth degree have been used to create expressions of how the quotient \( l_u/l_c \) is dependent on the quotient \( l_u/l_c \). The following expressions have been used to describe the relationship between \( l_u/l_c \) and \( l_u/l_c \), in the interval \( 0 \leq l_u/l_c \leq 1.5 \).

**Buckling:** \[
\frac{l_u}{l_c} = 0.0569 \left( \frac{l_u}{l_c} \right)^6 + 0.2133 \left( \frac{l_u}{l_c} \right)^5 - 1.6178 \left( \frac{l_u}{l_c} \right)^4 + 2.6533 \left( \frac{l_u}{l_c} \right)^3 - 1.0191 \left( \frac{l_u}{l_c} \right)^2 - 0.9667 \left( \frac{l_u}{l_c} \right) + 1.1
\]

**Moment:** \[
\frac{l_u}{l_c} = -0.2276 \left( \frac{l_u}{l_c} \right)^6 + 1.1533 \left( \frac{l_u}{l_c} \right)^5 - 2.1689 \left( \frac{l_u}{l_c} \right)^4 + 1.72 \left( \frac{l_u}{l_c} \right)^3 - 0.2236 \left( \frac{l_u}{l_c} \right)^2 - 0.462 \left( \frac{l_u}{l_c} \right) + 0.6
\]

**Horiz. stiff.** \[
\frac{l_u}{l_c} = -0.526 \left( \frac{l_u}{l_c} \right)^6 + 2.378 \left( \frac{l_u}{l_c} \right)^5 - 3.942 \left( \frac{l_u}{l_c} \right)^4 + 2.776 \left( \frac{l_u}{l_c} \right)^3 - 0.531 \left( \frac{l_u}{l_c} \right)^2 - 0.2678 \left( \frac{l_u}{l_c} \right) + 0.5
\]
Figure 12.4 Relationships between $l_u/l_c$ and $l_e/l_c$ for a pile in uniform soil, developed by Abendroth and Greimann (1989).

Figure 12.4 combined with the previous calculations give the results shown in Table 12.4 for the original soil model, and the results for the alternative soil model are shown in Table 12.5.

Table 12.4 Calculated embedded length $l_e$ and equivalent cantilever length $L_{equ}$, for the original soil model.

<table>
<thead>
<tr>
<th></th>
<th>No corrosion</th>
<th>2.4 mm corrosion allowance</th>
</tr>
</thead>
<tbody>
<tr>
<td>$l_u/l_c$</td>
<td>0.591</td>
<td>0.643</td>
</tr>
<tr>
<td>$l_e/l_c$</td>
<td>0.541</td>
<td>0.513</td>
</tr>
<tr>
<td>$L_{equ}$</td>
<td>4.979</td>
<td>4.675</td>
</tr>
<tr>
<td>Buckling</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$l_u/l_c$</td>
<td>0.413</td>
<td>0.408</td>
</tr>
<tr>
<td>$l_e/l_c$</td>
<td>1.815</td>
<td>1.647</td>
</tr>
<tr>
<td>$L_{equ}$</td>
<td>4.415</td>
<td>4.247</td>
</tr>
<tr>
<td>Moment</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$l_u/l_c$</td>
<td>0.397</td>
<td>0.397</td>
</tr>
<tr>
<td>$l_e/l_c$</td>
<td>1.748</td>
<td>1.603</td>
</tr>
<tr>
<td>$L_{equ}$</td>
<td>4.348</td>
<td>4.203</td>
</tr>
<tr>
<td>Horiz. stiff.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$l_u/l_c$</td>
<td>0.975</td>
<td>0.965</td>
</tr>
<tr>
<td>$l_e/l_c$</td>
<td>4.921</td>
<td>4.541</td>
</tr>
<tr>
<td>$L_{equ}$</td>
<td>5.521</td>
<td>5.141</td>
</tr>
</tbody>
</table>

Table 12.5 Calculated embedded length $l_e$ and equivalent cantilever length $L_{equ}$, for the alternative soil model.

<table>
<thead>
<tr>
<th></th>
<th>No corrosion</th>
<th>2.4 mm corrosion allowance</th>
</tr>
</thead>
<tbody>
<tr>
<td>$l_u/l_c$</td>
<td>0.119</td>
<td>0.128</td>
</tr>
<tr>
<td>$l_e/l_c$</td>
<td>0.975</td>
<td>0.965</td>
</tr>
<tr>
<td>$L_{equ}$</td>
<td>4.921</td>
<td>4.541</td>
</tr>
<tr>
<td>Buckling</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$l_u/l_c$</td>
<td>0.544</td>
<td>0.540</td>
</tr>
<tr>
<td>$l_e/l_c$</td>
<td>2.748</td>
<td>2.543</td>
</tr>
<tr>
<td>$L_{equ}$</td>
<td>3.348</td>
<td>3.143</td>
</tr>
<tr>
<td>Moment</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$l_u/l_c$</td>
<td>0.465</td>
<td>0.462</td>
</tr>
<tr>
<td>$l_e/l_c$</td>
<td>2.345</td>
<td>2.173</td>
</tr>
<tr>
<td>$L_{equ}$</td>
<td>2.945</td>
<td>2.773</td>
</tr>
<tr>
<td>Horiz. stiff.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Alternative pile cross-section

Figure 12:4 combined with the previous calculations for the alternative piles, HEM120, give the results shown in Table 12:6 and Table 12:7.

Table 12:6 Embedded length $l_e$ and equivalent cantilever length $L_{eq}$, original soil model and alternative pile.

<table>
<thead>
<tr>
<th></th>
<th>No corrosion</th>
<th>2.4 mm corrosion allowance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$l_e/l_c$</td>
<td>$L_{eq}$</td>
</tr>
<tr>
<td>Buckling</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$l_e$</td>
<td>$L_{eq}$</td>
</tr>
<tr>
<td></td>
<td>0.477</td>
<td>1.704</td>
</tr>
<tr>
<td>Horiz. stiff.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$l_e/l_c$</td>
<td>$L_{eq}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$l_e$</td>
<td>$L_{eq}$</td>
</tr>
<tr>
<td></td>
<td>0.401</td>
<td>1.432</td>
</tr>
<tr>
<td>Moment</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$l_e/l_c$</td>
<td>$L_{eq}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$l_e$</td>
<td>$L_{eq}$</td>
</tr>
<tr>
<td></td>
<td>0.395</td>
<td>1.411</td>
</tr>
<tr>
<td>Horiz. stiff.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$l_e/l_c$</td>
<td>$L_{eq}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$l_e$</td>
<td>$L_{eq}$</td>
</tr>
<tr>
<td></td>
<td>0.395</td>
<td>1.411</td>
</tr>
</tbody>
</table>

Table 12:7 Embedded length $l_e$ and equivalent cantilever length $L_{eq}$, alternative soil model and alternative pile.

<table>
<thead>
<tr>
<th></th>
<th>No corrosion</th>
<th>2.4 mm corrosion allowance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$l_e/l_c$</td>
<td>$L_{eq}$</td>
</tr>
<tr>
<td>Buckling</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$l_e$</td>
<td>$L_{eq}$</td>
</tr>
<tr>
<td></td>
<td>0.950</td>
<td>4.028</td>
</tr>
<tr>
<td>Moment</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$l_e/l_c$</td>
<td>$L_{eq}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$l_e$</td>
<td>$L_{eq}$</td>
</tr>
<tr>
<td></td>
<td>0.534</td>
<td>2.266</td>
</tr>
<tr>
<td>Horiz. stiff.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$l_e/l_c$</td>
<td>$L_{eq}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$l_e$</td>
<td>$L_{eq}$</td>
</tr>
<tr>
<td></td>
<td>0.458</td>
<td>1.942</td>
</tr>
<tr>
<td>Horiz. stiff.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$l_e/l_c$</td>
<td>$L_{eq}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$l_e$</td>
<td>$L_{eq}$</td>
</tr>
<tr>
<td></td>
<td>0.458</td>
<td>1.942</td>
</tr>
</tbody>
</table>
13 Loads and Rotations

The loads acting on an integral abutment bridge will result in normal pile forces, rotations and lateral movements of the abutments. Rotations of the abutments will give rise to small lateral displacements at the top of the piles. These displacements are added to the lateral thermal movements, see (13.1). The relationship between the lateral movements and the abutment rotations is illustrated in Figure 13:1.

\[
\Delta_{\text{tot}} = \frac{\Delta L_n}{2} + \theta \cdot H
\]  

(13.1)

The height \( H \) is the vertical distance between the level where the piles enter the abutment and up to the level where the center of gravity for the composite superstructure is located. The distance \( H \) is approximated to 1.65 m, which is based on the center of gravity for short term loading of the end girders.

The lateral movements are defined as positive when the length of the bridge are increasing, and the rotation angle are defined as positive clockwise in Figure 13:1. The rotations caused by traffic loads will always be positive in a single span bridge with this definition.

How a certain dead load affects the piles depends on in which order the different part of the bridge are mounted and cast at the construction site. The following construction order is assumed for the Leduån Bridge.

- Piles are driven
- Lower part of the abutment walls and the wingwalls are cast
- Steel girders are mounted, but are free to rotate until the top of the abutment wall shall be cast. Then they are fixed in the current position before they are embedded in concrete
- Bridge deck is cast in one stage, and the top of the abutment walls are cast last.
- Backfill behind the abutment walls
- Concrete pavement is cast and crash barriers are mounted.
With this construction order, the dead loads from the steel girders and the concrete deck slab are assumed to give no rotations or lateral movements at the top of the piles. While the steel girders are free to rotate at the supports until the top of the abutment walls are cast. Yet, the dead load from pavement and crash barriers will give rotations as well as lateral movements.

### 13.1 Permanent Loads

Dead loads from different parts of the bridge superstructure are calculated according to the Swedish Bridge Code - BRO2004. The following unit weights, $\gamma$, are taken from *BRO2004 21.11*.

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced concrete</td>
<td>25 kN/m$^3$</td>
</tr>
<tr>
<td>Steel</td>
<td>77 kN/m$^3$</td>
</tr>
</tbody>
</table>

#### 13.1.1 Girders

The weight and length of the steel girders are taken from the construction drawings of the Leduån Bridge, and are presented in *Table 13:1* together with their dead loads. The weights of the crossbeams, UPE300, are included in the girder weights. One crossbeam is included in each pair of end girders, and two crossbeams are included in the weight of the pair of midspan girders.

*Table 13:1* Steel girder weights, lengths and dead loads.

<table>
<thead>
<tr>
<th>Weight [kg]</th>
<th>L [m]</th>
<th>q [kN/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 640</td>
<td>11</td>
<td>9.7</td>
</tr>
<tr>
<td>19 256</td>
<td>18</td>
<td>10.7</td>
</tr>
<tr>
<td>10 640</td>
<td>11</td>
<td>9.7</td>
</tr>
<tr>
<td>40 536</td>
<td>40</td>
<td></td>
</tr>
</tbody>
</table>

#### 13.1.2 Deck

The cross-section area of the concrete deck slab is 1.5 m$^2$.

$$q_{\text{deck}} = A_{\text{deck}} \cdot \gamma_{\text{conc}}$$

$$q_{\text{deck}} = 1.5 \cdot 25 = 37.5 \text{ kN/m}$$

#### 13.1.3 Pavement

The pavement on top of the bridge deck is a 95 mm thick layer of net reinforced concrete.

$$q_{\text{pave}} = A_{\text{pave}} \cdot \gamma_{\text{conc}}$$

$$q_{\text{pave}} = (4.850 \cdot 0.095) \cdot 25 = 11.5 \text{ kN/m}$$

#### 13.1.4 Crash Barriers

$$q_{\text{barr}} = 0.5 \text{ kN/m}$$
13.1.5 Concrete Shrinkage

The concrete in the bridge deck slab will shrink and cause rotations at the supports. The normal forces at the supports will not be affected since the example bridge is a single span bridge.

The bridge deck slab is made of concrete C40/50. The concrete shrinkage will give rise to a normal force in a perpendicular cross-section of the deck slab. This force is assumed to act in the bridge deck’s centre of gravity, and will not coincide with the centre of gravity for the composite cross-section. Therefore, an additional bending moment will act on the superstructure. The cross-section calculations are presented in Appendix J.

The Swedish Regulations for Concrete Structures - BBK04 Chapter 2.4.6 gives the mean value of the final shrinkage, $\varepsilon_{cs} = 0.25 \cdot 10^{-3}$, a relative humidity of 75 % is assumed. A fictitious stress can be calculated according to Hooke’s law, long-term loading gives $\phi = 2$. Material reduction factors are not used in the calculations, since it would give an underestimation of the stresses.

\[
\sigma_{\text{shrink}} = \frac{E}{(1 + \phi)} \varepsilon_{cs} \quad (13.4)
\]

\[\sigma_{\text{shrink}} = 0.25 \cdot 10^{-3} \cdot \frac{35 \cdot 10^9}{(1 + 2)} = 2.917 \, \text{MPa}\]

The cross section area of the concrete deck is 1.5 $m^2$, which gives a normal force with the following magnitude,

\[
F_{\text{shrink}} = \sigma_{\text{shrink}} \cdot A_{\text{deck}} \quad (13.5)
\]

\[F_{\text{shrink}} = 2.917 \cdot 1.5 = 4.375 \, \text{MN}.
\]

The uniformly distributed moment, due to the shrinkage, are calculated by multiplying the normal force with the lever arm from the centre of the deck to the centre of gravity for the whole cross section. The dimensions of the midspan girders are used in the calculations since they have a lower centre of gravity than the end girders.

\[
M_{\text{shrink}} = F_{\text{shrink}} \cdot (e_{LT} - e_{\text{deck}}) \quad (13.6)
\]

\[M_{\text{shrink}} = 4.38 \cdot (0.512 - (-0.122)) = 2.774 \, \text{MNm}\]

13.1.6 Summary of Permanent Loads

Dead loads from the steel girders and the concrete deck are assumed to only give normal forces in the piles, and no rotations. This assumption is made since their weights are loaded on the piles when the superstructure is still behaving as simply supported. The loads from pavement, crash barriers, and concrete shrinkage will be loaded on the piles after the superstructure has been integrated in the abutments. They will therefore give contributions to the rotations at the top of the piles, as well as lateral displacements. Figure 13.2 illustrates how the dead loads are distributed over the length of the bridge.
The permanent loads are uniformly distributed over the cross section, and there are no eccentric dead loads. The resulting pile forces from the permanent loads are the same in all piles, because of the evenly distributed dead loads. Table 13:2 shows the normal force in a single pile due to dead loads.

### Table 13:2 Normal forces in a single pile due to the dead loads.

<table>
<thead>
<tr>
<th>Dead Load</th>
<th>Normal Force [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel girders</td>
<td>33.83</td>
</tr>
<tr>
<td>Concrete deck</td>
<td>125.00</td>
</tr>
<tr>
<td>Crash barriers</td>
<td>1.67</td>
</tr>
<tr>
<td>Pavement</td>
<td>38.33</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>198.8</strong></td>
</tr>
</tbody>
</table>

13.1.7 Rotations due to Permanent Loads

The gravity loads that are acting on the superstructure are transformed into end rotations, $\theta_w$, assuming a simply supported superstructure. The connection between the superstructure and the abutment wall are treated as totally rigid. The relationship between the bending stiffness, $W$, of the superstructure and the bending stiffness of the six piles in each abutment is about 140:1 for long term loading, and larger than 400:1 for short term loading. Hence, the piles are assumed to not give any resistance to the rotations and lateral displacements of the abutments. These assumptions give rotations at the top of the piles that equals $\theta_w$. A uniformly distributed load, $q$, gives the following rotation

$$\theta_w = \frac{qL_o^3}{24EI_{LT}},$$  \hspace{1cm} (13.7)
where \( L_b \) and \( I_{LT} \) are the length respectively the long term moment of inertia for the superstructure. Dead loads from pavement and crash barriers, giving rotations of the abutments, are both uniformly distributed and give the rotations that are presented in Table 13:3. The concrete shrinkage will also give a rotation of the abutments. In this calculation it is assumed that the shrinkage process take place instantaneously. The contribution from the shrinkage to the abutment rotation can be calculated as following

\[
\theta_{\text{shrink}} = \frac{ML_b}{3EI_{LT}}.
\]  

(13.8)

Table 13:3 Rotations caused by dead loads and concrete shrinkage.

<table>
<thead>
<tr>
<th></th>
<th>( \theta ) [%e rad]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pavement</td>
<td>1.48</td>
</tr>
<tr>
<td>Crash barriers</td>
<td>0.06</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>2.48</td>
</tr>
</tbody>
</table>

13.2 Variable Loads

Expressions for pile forces, rotations, and lateral displacements are formulated for three types of variable loads in order to calculate the strain in the piles. The following variable loads are taken into account.

- Traffic loads (gravity induced)
- Loads due to lateral thermal displacements of the abutments
- Loads induced by temperature gradients in the superstructure

13.2.1 Thermal Loads

Thermal loads have been presented in detail in a previous chapter, and are therefore just mentioned briefly in this chapter.

Thermal loads are separated into two types of temperature changes. The first one is the uniform temperature variations in the bridge superstructure, which results in a varying length of the bridge and lateral displacements of the abutments. The second is the vertical temperature gradient, or the temperature difference between steel girders and concrete deck.

13.2.1.1 Uniform Temperature Variations – Changes in EBT

The characteristic maximum and minimum temperatures that occur once in fifty years can be used for design purpose in order to estimate how large temperature displacement that can be expected. If the pile fatigue is studied, it might be better to use a simulation of the temperature in order to take both the daily and annual variations into consideration.

Calculations of the maximum lateral displacements are performed to be able to evaluate if the strains in the piles will exceed the yield strain. The bridge is assumed to be located in Karesuando in these calculations, since it is the studied location with the largest temperature variation.
variations. The temperatures that have been used are the maximum and minimum shade air temperatures given in BRO2004. The shade air temperatures are transformed to EBTs, according to equation (10.6). The bridge deck is assumed to be cast at a day with an air temperature of +10°C.

\[ T_{\text{eff \ max/min}} = 4 + 0.98 \cdot T_{\text{max/min}} \]  

\[ T_{\text{max}} = 31^\circ\text{C} \quad T_{\text{eff max}} = 4 + (0.98 \cdot 31) = 34.4^\circ\text{C} \]

\[ T_{\text{min}} = -48^\circ\text{C} \quad T_{\text{eff min}} = 4 + (-0.98 \cdot 48) = -43.0^\circ\text{C} \]

\[ \Delta_{\text{abut}} = \frac{\alpha \cdot \Delta T_y \cdot L_b}{2} \]  

\[ L_b = 40 \text{ m} \]

\[ \alpha = 10^{-5} \text{ C}^{-1} \]

\[ \Delta_{\text{abut}}^+ = \frac{10^{-5} \cdot (34.4 - 10) \cdot 40}{2} = 4.9 \text{ mm} \]

\[ \Delta_{\text{abut}}^- = \frac{10^{-5} \cdot (-43.0 - 10) \cdot 40}{2} = -10.6 \text{ mm} \]

The moment that is induced in the top of the piles can be calculated by the following equation

\[ M_T = \frac{6E_p I_p \Delta_{\text{abut}}}{L_{\text{equ},h}} \]  

13.2.1.2 Varying Temperature Gradient

The temperature differences in the superstructure are modelled as a sharp change in temperature in the joint between the concrete deck and the steel girders. This model does not reflect the real situation entirely, since there would not be any steps in the vertical temperature curve. Instead it would be a temperature gradient which would be dependent on the vertical depth and different weather conditions. This simplification of the temperature differences is done in order to keep the model quite simple. Since the temperature distribution curve does not coincide with the centre of gravity for the cross-section, a secondary bending moment will occur, see Figure 13:3.
The end rotations of the bridge girders, $\theta_{\Delta T}$, are calculated assuming a simply supported superstructure. The rotations at the top of the piles are then assumed to be equal to $\theta_{\Delta T}$, since the connection between the bridge girders and the abutments is modelled as totally rigid. The end girders’ moment of inertia is lowest, and is taken as valid for the whole superstructure.

$$\theta_{\Delta T} = \frac{M_{\Delta T} \cdot L_0}{3EI_{\text{girders}}}$$  \hspace{1cm} (13.12)

$$\varepsilon_{\Delta T, \text{girders}} = \alpha \cdot \Delta T_{\text{TG}}$$  \hspace{1cm} (13.13)

$$\sigma_{\Delta T, \text{girders}} = \varepsilon_{\Delta T} \cdot E_{\text{steel}}$$  \hspace{1cm} (13.14)

$$F_{\Delta T, \text{girders}} = \sigma_{\Delta T} \cdot A_{\text{steel}}$$  \hspace{1cm} (13.15)

$$M_{\Delta T, \text{girders}} = F_{\Delta T} \cdot (\varepsilon_{\Delta T} - e_{\text{ST}})$$  \hspace{1cm} (13.16)

$$\alpha = 1.0 \cdot 10^{-5} \, ^{\circ}\text{C}^{-1}$$

$$E_{\text{steel}} = 210 \, \text{GPa}$$

$$A_{\text{steel}} = 0.1143 \, \text{and} \, 0.1291 \, \text{m}^2 \, \text{(end girders respectively midspan girders)}$$

$$e_{\text{ST}} = 210 \, \text{and} \, 251 \, \text{mm} \, \text{(end girders respectively midspan girders)}$$

$$e_{\text{steel}} = 816 \, \text{and} \, 852 \, \text{mm} \, \text{(end girders respectively midspan girders)}$$

The characteristic values of the positive respectively negative temperature gradients in a composite bridge are, according to ENV1991-2-5, +15°C respectively -18°C. The Swedish Bridge Code - BRO2004 recommends +10°C and -5°C. Yet, if a study of the pile fatigue shall be done, then a simulation of the varying temperature gradient is preferable. The simulation process has been described in Section 11.2.

The maximum rotations, due to the temperature differences, that are expected to occur 2 times during the bridge service lifetime of 120 years, are calculated according to the characteristic values from ENV1991-2-5.

$$\theta_{\Delta T} = \frac{-\alpha \cdot \Delta T_{\text{TG}} \cdot A_{\text{steel}} \cdot (e_{\text{steel}} - e_{\text{ST}}) \cdot L_0}{3 \cdot I_{\text{ST}}}$$  \hspace{1cm} (13.17)

$$\Delta T_{TG}^+ = 15 \, ^{\circ}\text{C}$$

$$\theta_{\Delta T} = \frac{-10^{-5} \cdot 15 \cdot 0.1143 \cdot (0.816 - 0.210) \cdot 40}{3 \cdot 0.0987} = -0.001404$$

$$\Delta T_{TG}^- = -18 \, ^{\circ}\text{C}$$

$$\theta_{\Delta T} = \frac{-10^{-5} \cdot (-18) \cdot 0.1143 \cdot (0.816 - 0.210) \cdot 40}{3 \cdot 0.0987} = 0.001684$$
13.2.2 Traffic Loads

*Fatigue load according to BRO2004*

A bridge shall be designed to resist the fatigue load illustrated in Figure 13:4. If the number of vehicles that daily crosses the bridge exceeds 10 000, then $4 \cdot 10^5$ load cycles shall be assumed. If there are less than 10 000 crossing per day, $10^5$ load cycles shall be assumed.

![Figure 13:4 Illustration of the fatigue load according to BRO2004.](image)

The maximum normal force at one of the abutments and the maximum rotation of the same abutment will not coincide. Therefore two calculations are done, one with the maximum normal force and one with the maximum rotation.

**Maximum normal force**

The maximum value of the resulting normal force on the abutment, $R$, will occur at the time when all of the axels of a vehicle have entered the bridge. The maximum resulting force on the abutments is calculated as following

$$R_{\text{left}} = \sum_{i} N_{\text{axel},i} \cdot \left( L_b - a_i \right) \frac{L_b}{L_b}$$  \hspace{1cm} (13.18)

$$R_{\text{right}} = \sum_{i} N_{\text{axel},i} \cdot a_i \frac{L_b}{L_b}$$  \hspace{1cm} (13.19)

where $a_i$ is the distance from the left abutment to the $i$:th load. The maximum normal force will in this case be

$$R_{\text{max, fatigue}} = 601.9 \text{ kN}$$

The vehicle loads in the fatigue calculations are eccentric, and the transverse moment caused by this eccentricity has to be calculated.

$$M_{\text{fatigue}} = e_{\text{fatigue}} \cdot R_{\text{max, fatigue}}$$  \hspace{1cm} (13.20)

$$M_{\text{fatigue}} = 1.000 \cdot 601.9 = 601.9 \text{ kNm}$$
**Maximum pile force due to the traffic fatigue load**

The traffic fatigue load is eccentric, this gives unevenly distributed normal forces in the piles. The eccentric vertical load that is acting on the abutment is replaced by a vertical force acting on the symmetric axis, and a moment around the symmetric axis, see Figure 13:5.

![Figure 13:5](image)

Figure 13:5 Eccentric load is replaced by a force and moment acting at the symmetric axis.

The eccentric load is distributed over the piles as shown in Figure 13:6. Equation (13.22) can be used to calculate the maximum force that will act in one of the outer piles, *Pile 1* in Figure 13:6.

![Figure 13:6](image)

Figure 13:6 Distributions of normal forces in the piles due to eccentric loads.

\[
N_{\text{pile},i} = \frac{R}{n} + \frac{M \cdot x_i}{\sum_{i=1}^{n} x_i^2} \tag{13.21}
\]

\[
N_{\text{pile}}^{\text{max}} = \frac{R}{n} + \frac{M \cdot x_{\text{max}}}{\sum_{i=1}^{n} x_i^2} \tag{13.22}
\]

\[
N_{\text{fatigue}}^{\text{max}} = \frac{601.9}{6} + \frac{601.9 \cdot 2.1}{12.7} = 199.8 \text{ kN}
\]
The rotation of the abutments are calculated as following

\[
\theta_{\text{left}} = \sum_{i=1}^{n} \left( \frac{N_{\text{axel,i}} \cdot L_{a_i} \cdot (L_{b} - a_i)}{6EI_{ST}} \left( 1 - \frac{(L_{b} - a_i)^2}{L_{b}^2} \right) \right) \tag{13.23}
\]

\[
\theta_{\text{right}} = \sum_{i=1}^{n} \left( \frac{N_{\text{axel,i}} \cdot L_{a_i} \cdot a_i}{6EI_{ST}} \left( 1 - \frac{a_i^2}{L_{b}^2} \right) \right) \tag{13.24}
\]

When the maximum normal force occurs in one of the abutments, then the heaviest loaded pile will have the following rotation at the top.

\[\theta_{\text{fatigue}} = 0.001077\]

**Maximum rotation**

The rotations of the abutments, \(\theta\), are calculated according to (13.23) and (13.24). The maximum rotation of one of the abutments occurs when the last axel of the fatigue vehicle are 17.774 m from the left abutment. Assuming that, the vehicle is travelling to the right in Figure 13.4.

\[\theta_{\text{max,fatigue}} = 0.003126\]

The normal force, due to the traffic fatigue load, that acts on the abutment at the same time as the rotations reaches its maximum value is calculated according to (13.18) and (13.19). The calculations of transverse moment and resulting pile forces are done according to (13.20)-(13.22).

\[R_{\text{fatigue}} = 375.4 \text{ kN}\]

\[M_{\text{fatigue}} = 1.000 \cdot 375.4 = 375.4 \text{ kNm}\]

\[N_{\text{fatigue, pile}} = \frac{375.4}{6} + \frac{375.4 \cdot 2.1}{12.7} = 124.6 \text{ kN}\]
13.2.3 Backfill Soil Pressure

Soil pressures are in this report calculated according to BRO2004. Figure 13:7 illustrates the load situation, and equation (13.25) - (13.27) are used to estimate the soil pressures.

\[ p = p_{act} \quad \text{if} \quad \Delta_{abut} = 0 \]  
\[ p = p_{act} + c_1 \cdot \Delta_{abut} \cdot \frac{200}{H_{bw}} \cdot (p_{pass} - p_{act}) \quad \text{if} \quad 0 < \Delta_{abut} < (H_{bw}/200) \]  
\[ p = p_{act} + c_1 \cdot (p_{pass} - p_{act}) \quad \text{if} \quad \Delta_{abut} \geq (H_{bw}/200) \]  

\[ c_1 = 0.5 \quad \text{ (soil pressure is favourable for the piles)} \]

The lateral thermal movement of the abutment backwall, \( \Delta_{abut} \), shall be taken as the lateral movement between the lowest and the highest temperature, according to BRO2004 21.231. The soil pressure is assumed to give a reduction of the abutment rotations. The lateral displacements due to annual temperature variations are assumed to be independent of the soil pressure. The calculations of the reduction of the rotation angle due to the soil pressure are shown below.

- \( H_{bw} = 3.00 \text{ m} \) height of the abutment backwall
- \( B = 5.00 \text{ m} \) width of the abutment backwall
- \( \gamma = 18 \text{ kN/m}^3 \) unit weight of the backfill
- \( K_p = 5.83 \) passive soil pressure coefficient
- \( K_a = 0.34 \) active soil pressure coefficient

\[ p_{pass} = K_p \cdot \gamma \cdot H_{bw} \]  
\[ p_{pass} = 5.83 \cdot 18 \cdot 3.0 \]  
\[ p_{pass} = 314.82 \text{ kPa} \]

\[ p_{act} = K_a \cdot \gamma \cdot H_{bw} \]  
\[ p_{act} = 0.34 \cdot 18 \cdot 3.0 \]  
\[ p_{act} = 18.36 \text{ kPa} \]

\[ H_{bw}/200 = 3000/200 = 15 \text{ mm} \quad \Rightarrow \quad \Delta_{abut} \geq (H_{bw}/200) \]
\( p = 18.36 + 0.5 \cdot (314.82 - 18.36) \)
\( p = 166.6 \text{ kPa} \)

\[
F_{\text{res}} = H_{bw} \cdot B \cdot (p/2) \quad (13.30)
\]

\[
F_{\text{res}} = 3 \cdot 5 \cdot (166.6/2) \]
\( F_{\text{res}} = 1.250 \text{ MN} \)

\[
M_{\text{soil}} = F_{\text{res}} \cdot \left( \frac{2H_{bw}}{3} - e_{ST}^{\text{adj}} \right)
\]

\( e_{ST}^{\text{adj}} = 0.75 \)  The superstructure’s centre of gravity, adjusted to a coordinate system with the zero level at the top of the abutment backwall and positive downwards.

\[
M_{\text{soil}} = 1.250 \cdot \left( \frac{2 \cdot 3}{3} - 0.75 \right) \]
\( M_{\text{soil}} = 1.562 \text{ MNm} \)

\[
\theta_{\text{soil}} = -\frac{M_{\text{soil}} \cdot L_h}{2 \cdot E I_{ST}}, \quad (13.31)
\]

\[
\theta_{\text{soil}} = -\frac{1.562 \cdot 10^6 \cdot 40}{2 \cdot 210 \cdot 10^9 \cdot 0.0987}, \quad \theta_{\text{soil}} = -0.00151
\]

\[
\varepsilon_{\text{pile,soil}} = \left( \frac{2 \cdot \varnothing}{L_{\text{equ,m}}} \right) \cdot \theta_{\text{soil}} \quad (13.32)
\]

\[
\begin{array}{ll}
\text{No corrosion} & \text{Corrosion 2.4 mm} \\
\varepsilon_{\text{soil}} = \left( \frac{2 \cdot 0.1683}{4.415} \right) \cdot 0.00151 & \varepsilon_{\text{pile,soil}} = \left( \frac{2 \cdot 0.1635}{4.247} \right) \cdot 0.00151 \\
\varepsilon_{\text{pile}} = -0.000115 & \varepsilon_{\text{soil}} = -0.000112
\end{array}
\]

The moment caused by the backfill soil pressure would reduce the pile strains up to approximately 5% of the yield strain. The soil pressure is assumed to reduce or eliminate small rotations and translations due to traffic loads and daily temperature variations. This would be favourable for the piles, and the effect of the backfill are brought into the calculations of the total piles strain as shown in (13.33).

\[
\varepsilon_{\text{pile}} = \left( \frac{3 \cdot \varnothing}{L_{\text{equ,h}}^2} \right) \cdot H \cdot \theta_{\text{soil}} + \left( \frac{2 \cdot \varnothing}{L_{\text{equ,m}}} \right) \cdot \theta_{\text{soil}} \quad (13.33)
\]
13.3 Summary of Forces, Movements and Rotations

**Dead loads**

<table>
<thead>
<tr>
<th></th>
<th>( N_{\text{dead}}^{\text{pile}} ) [kN]</th>
<th>( \theta_{\text{dead}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel girders</td>
<td>33.83</td>
<td>-</td>
</tr>
<tr>
<td>Concrete deck</td>
<td>125.00</td>
<td>-</td>
</tr>
<tr>
<td>Crash barriers</td>
<td>1.67</td>
<td>0.00006</td>
</tr>
<tr>
<td>Pavement</td>
<td>38.33</td>
<td>0.00148</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>-</td>
<td>0.00248</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>198.83</td>
<td>0.00402</td>
</tr>
</tbody>
</table>

**Lateral thermal movements**

\[ \Delta_{\text{abut}}^+ = 4.9 \, \text{mm} \]
\[ \Delta_{\text{abut}}^- = -10.6 \, \text{mm} \]

**Temperature gradient**

\[ \Delta T^+ = 15 \, ^\circ \text{C} \]
\[ \Delta T^- = -18 \, ^\circ \text{C} \]
\[ \theta_{\Delta T}^+ = -0.001404 \]
\[ \theta_{\Delta T}^- = 0.001684 \]

**Traffic loads**

**Maximum normal force**

\[ N_{\text{max}}^{\text{pile}} = 199.8 \, \text{kN} \]
\[ N_{\text{pile}} = 124.6 \, \text{kN} \]
\[ \theta_{\text{fatigue}} = 0.001077 \]
\[ \theta_{\text{fatigue}}^{\text{max}} = 0.003126 \]

**Backfill soil pressure**

Maximum reduction of positive rotations

\[ \theta_{\text{soil}} = -0.00151 \]
14 Maximum Strain Calculations

The maximum expected strains in the top of a pile during an annual cycle, are calculated as the sum of the strains caused by the following loads and movements.

- Lateral thermal movements
- Temperature gradients
- Traffic loads
- Dead loads

The strain variation at the top of the pile is studied at the two points that will experience the largest strain variations, P₁ and P₂, see Figure 14:1.

The strain amplitudes will be the same at both points, due to the symmetry, even if the maximum and minimum strains would be different. For instance, it is possible that P₂ will experience only compressive strains which would be rather harmless in the development of cracks due to the fatigue. But, P₂ will also experience the highest strains, and the yield strain of about 0.002 would first be reached at this point. It would not be that relevant for the fatigue calculations if only the first annual cycle gives plastic deformations. It would only change the equilibrium position for the piles. Low-cycle fatigue would only be a major problem if there would be repeating cycles giving strain differences more than $2\varepsilon_y$, see Figure 14:2a. That situation is not expected, it seems more likely that the stress-strain relation would be varying as shown in Figure 14:2b.

![Figure 14:1](image1.png)  
![Figure 14:2a](image2a.png)  
![Figure 14:2b](image2b.png)

**Figure 14:1** Pile strains are studied at the points P₁ and P₂.

**Figure 14:2** Illustration of stress-strain relationship for two possible situations, elastic perfectly plastic material.
The pile strains are calculated as a function of the lateral movements, normal forces and rotations that occur. Normal forces and strains are defined as positive in compression, see Figure 14:3.

According to the previous definitions, strains in P₁ and P₂ are calculated according to (14.1) and (14.2). Two different equivalent cantilever lengths are used in the calculations. The pile strains due to lateral displacements are based on the horizontal pile stiffness model, and \( L_{equ,h} \) is used. The strains from the rotations due to gravity loads and temperature gradients are based on the maximum moment equivalency model, and \( L_{equ,m} \) is used. How these equivalent cantilever lengths are calculated and why different lengths are used is presented in section 12.3 and Appendix B.

\[
\varepsilon_{P_1} = \frac{N_{dead,pile} + N_{traffic,pile}}{A_{pile} \cdot E} \left( \Delta_{hub} + H \left( \theta_{\alpha_T} + \theta_{traffic} + \theta_{dead} \right) \right) - \frac{3 \cdot \phi}{L_{equ,h}} \left( \theta_{\alpha_T} + \theta_{traffic} + \theta_{dead} \right) \tag{14.1}
\]

\[
\varepsilon_{P_2} = \frac{N_{dead,pile} + N_{traffic,pile}}{A_{pile} \cdot E} \left( \Delta_{hub} + H \left( \theta_{\alpha_T} + \theta_{traffic} + \theta_{dead} \right) \right) + \frac{3 \cdot \phi}{L_{equ,m}} \left( \theta_{\alpha_T} + \theta_{traffic} + \theta_{dead} \right) \tag{14.2}
\]

The following fatigue calculations are based on the strains in \( P_2, \) (14.2). Yet, same result would be achieved if \( P_1, \) (14.1), was used instead. If H-piles are studied then \( \phi \) is replaced by \( b. \)
14.1 Maximum Positive Lateral Movement - Summertime

The traffic load used in the calculations is the traffic fatigue load model according to BRO2004. Dead loads and traffic loads will give extra lateral movements in the same direction as the thermal movement, in summertime. The most ultimate temperature gradient is also assumed to occur at the warmest day, in order to illustrate the worst situation. The most negative temperature gradient that statistically occurs once during fifty years is -18°C, according to ENV1991-2-5. It is not realistic to assume that a negative gradient of this size would occur at the warmest day of the year. Yet, in these calculations it is assumed to be possible, just to illustrate the worst case. The positive effect of the backfill soil pressure is not taken into consideration in the calculations of maximum strains.

No corrosion

<table>
<thead>
<tr>
<th></th>
<th>Maximum normal force</th>
<th>Maximum rotation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.1683 m</td>
<td>Δ쑥abut = 0.0049 m</td>
</tr>
<tr>
<td>Apile</td>
<td>4973.1 mm²</td>
<td>θ_TA = 0.001684</td>
</tr>
<tr>
<td>H</td>
<td>1.65 m</td>
<td>θ_traffic = 0.001077</td>
</tr>
<tr>
<td>Lequ.m</td>
<td>4.415 m</td>
<td>θ_dead = 0.004024</td>
</tr>
<tr>
<td>Lequ.h</td>
<td>4.348 m</td>
<td>N_tot = 398.63 kN</td>
</tr>
</tbody>
</table>

εP2 = 0.0001330

Corrosion 2.4 mm

<table>
<thead>
<tr>
<th></th>
<th>Maximum normal force</th>
<th>Maximum rotation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.1635 m</td>
<td>Δ㎏abut = 0.0049 m</td>
</tr>
<tr>
<td>Apile</td>
<td>3722.3 mm²</td>
<td>θ_TA = 0.001684</td>
</tr>
<tr>
<td>H</td>
<td>1.65 m</td>
<td>θ_traffic = 0.001077</td>
</tr>
<tr>
<td>Lequ.m</td>
<td>4.247 m</td>
<td>θ_dead = 0.004024</td>
</tr>
<tr>
<td>Lequ.h</td>
<td>4.203 m</td>
<td>N_tot = 398.63 kN</td>
</tr>
</tbody>
</table>

εP2 = 0.0001480

Alternative soil model

<table>
<thead>
<tr>
<th></th>
<th>Maximum normal force</th>
<th>Maximum rotation</th>
</tr>
</thead>
<tbody>
<tr>
<td>No corrosion</td>
<td>Lequ.m = 3.348 m</td>
<td>εP2 = 0.002002</td>
</tr>
<tr>
<td></td>
<td>Lequ.h = 2.945 m</td>
<td>εP2 = 0.002333</td>
</tr>
<tr>
<td>Corrosion 2.4 mm</td>
<td>Lequ.m = 3.143 m</td>
<td>εP2 = 0.002244</td>
</tr>
<tr>
<td></td>
<td>Lequ.h = 2.773 m</td>
<td>εP2 = 0.002576</td>
</tr>
</tbody>
</table>

Alternative pile cross-section

Original soil model

<table>
<thead>
<tr>
<th></th>
<th>Maximum normal force</th>
<th>Maximum rotation</th>
</tr>
</thead>
<tbody>
<tr>
<td>No corrosion</td>
<td>Lequ.m = 4.032 m</td>
<td>εP2 = 0.001089</td>
</tr>
<tr>
<td></td>
<td>Lequ.h = 4.011 m</td>
<td>εP2 = 0.001242</td>
</tr>
<tr>
<td>Corrosion 2.4 mm</td>
<td>Lequ.m = 3.885 m</td>
<td>εP2 = 0.001206</td>
</tr>
<tr>
<td></td>
<td>Lequ.h = 3.875 m</td>
<td>εP2 = 0.001341</td>
</tr>
</tbody>
</table>

Alternative soil model

<table>
<thead>
<tr>
<th></th>
<th>Maximum normal force</th>
<th>Maximum rotation</th>
</tr>
</thead>
<tbody>
<tr>
<td>No corrosion</td>
<td>Lequ.m = 2.866 m</td>
<td>εP2 = 0.001825</td>
</tr>
<tr>
<td></td>
<td>Lequ.h = 2.542 m</td>
<td>εP2 = 0.002149</td>
</tr>
<tr>
<td>Corrosion 2.4 mm</td>
<td>Lequ.m = 2.667 m</td>
<td>εP2 = 0.002047</td>
</tr>
<tr>
<td></td>
<td>Lequ.h = 2.376 m</td>
<td>εP2 = 0.002377</td>
</tr>
</tbody>
</table>
14.2 Maximum Negative Lateral Movement - Wintertime

Negative lateral movements can be observed in the wintertime, when bridges contracts due to low temperatures. Dead loads and traffic loads will in this case decrease the negative lateral movements and the rotations, and will not contribute to a wider strain range. The lowest strains, giving the largest strain range, would be achieved when no vehicles are on the bridge. A positive temperature gradient, +15°C, that statistically occurs once during fifty years is used in the calculations.

<table>
<thead>
<tr>
<th>No corrosion</th>
<th>No traffic load</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varnothing$</td>
<td>0.1683 m</td>
</tr>
<tr>
<td>$A_{\text{pile}}$</td>
<td>4973.1 mm$^2$</td>
</tr>
<tr>
<td>$H$</td>
<td>1.65 m</td>
</tr>
<tr>
<td>$L_{\text{equ.m}}$</td>
<td>4.415 m</td>
</tr>
<tr>
<td>$L_{\text{equ.h}}$</td>
<td>4.348 m</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>$\Delta_{\text{abut}}$</th>
<th>$\theta_{\text{AT}}$</th>
<th>$\theta_{\text{traffic}}$</th>
<th>$\theta_{\text{dead}}$</th>
<th>$N_{\text{tot}}$</th>
<th>$\varepsilon_{P2}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>-0.0106 m</td>
<td>-0.001404</td>
<td>0</td>
<td>0.004024</td>
<td>198.83 kN</td>
<td>0.000223</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Corrosion 2.4 mm</th>
<th>No traffic load</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varnothing$</td>
<td>0.1627 m</td>
</tr>
<tr>
<td>$A_{\text{pile}}$</td>
<td>3722.3 mm$^2$</td>
</tr>
<tr>
<td>$H$</td>
<td>1.65 m</td>
</tr>
<tr>
<td>$L_{\text{equ.m}}$</td>
<td>4.247 m</td>
</tr>
<tr>
<td>$L_{\text{equ.h}}$</td>
<td>4.203 m</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>$\Delta_{\text{abut}}$</th>
<th>$\theta_{\text{AT}}$</th>
<th>$\theta_{\text{traffic}}$</th>
<th>$\theta_{\text{dead}}$</th>
<th>$N_{\text{tot}}$</th>
<th>$\varepsilon_{P2}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>-0.0106 m</td>
<td>-0.001404</td>
<td>0</td>
<td>0.004024</td>
<td>198.83 kN</td>
<td>0.000282</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Alternative soil model</th>
<th>No traffic Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>No corrosion</td>
<td>$L_{\text{equ.m}} = 3.348 m$</td>
</tr>
<tr>
<td></td>
<td>$L_{\text{equ.h}} = 2.945 m$</td>
</tr>
<tr>
<td>Corrosion 2.4 mm</td>
<td>$L_{\text{equ.m}} = 3.143 m$</td>
</tr>
<tr>
<td></td>
<td>$L_{\text{equ.h}} = 2.773 m$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Alternative pile cross-section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original soil model</td>
</tr>
<tr>
<td>No corrosion $L_{\text{equ.m}} = 4.032 m$</td>
</tr>
<tr>
<td>$L_{\text{equ.h}} = 4.011 m$</td>
</tr>
<tr>
<td>Corrosion 2.4 mm $L_{\text{equ.m}} = 3.885 m$</td>
</tr>
<tr>
<td>$L_{\text{equ.h}} = 3.875 m$</td>
</tr>
</tbody>
</table>

<p>| Alternative soil model         |</p>
<table>
<thead>
<tr>
<th>No traffic Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>No corrosion $L_{\text{equ.m}} = 2.866 m$</td>
</tr>
<tr>
<td>$L_{\text{equ.h}} = 2.542 m$</td>
</tr>
<tr>
<td>Corrosion 2.4 mm $L_{\text{equ.m}} = 2.667 m$</td>
</tr>
<tr>
<td>$L_{\text{equ.h}} = 2.376 m$</td>
</tr>
</tbody>
</table>
14.3 Discussion About the Strain Calculations

14.3.1 Pipe Piles

**Original soil model**
The piles are made of steel with yield strength of 440 MPa (S440J2H), this gives a yield strain of about 0.0021. The calculated strains, according to the equivalent cantilever method, are all lower than the yield strain, and there would not be any plastic deformations of the piles. The maximum strain range is $0.61 \cdot \varepsilon_y$ for piles unaffected by corrosion, and $0.65 \cdot \varepsilon_y$ for piles that have corroded to a depth of 2.4 mm. This is about one third of the strain range, $2 \cdot \varepsilon_y$, which would be necessary to achieve an annual cycle which would give plastic deformations every year. A low-cycle fatigue failure would not be expected according to these calculations, but it is still necessary to perform an ordinary fatigue calculation to prove that there would not be any high-cycle fatigue failure. A short analyse is also made to get an idea of how long a similar bridge can be, before low-cycle fatigue become a possible failure mode.

**Alternative soil model**
The alternative soil model gives compressive pile strains which exceed the yield strain. The piles are made of steel pipes with cross-section class 1, which means that their rotation capacity enables plastic hinges. The maximum strain range is $1.07 \cdot \varepsilon_y$ for piles unaffected by corrosion, and $1.17 \cdot \varepsilon_y$ for piles that have corroded to a depth of 2.4 mm. Only the outmost part of the pile cross-section that is in compression will experience stresses which exceed the yield strength. The highest tensile stress at the opposite side of the pile is about 70% of the yield strength. There will be no reversible cycles with amplitudes of $2 \cdot \varepsilon_y$. The cycles with the largest amplitudes will still only be about 58% of $2 \cdot \varepsilon_y$. The stress-strain situation for an elastic perfectly plastic material could be simplified as shown in Figure 14:4.

![Figure 14:4 Illustration of stress-strain relationship, elastic perfectly plastic material.](image_url)

The effect of higher traffic loads has also been studied, with the intention of finding out whether or not it is realistic to believe that cyclic plastic deformations would appear in the piles supporting the abutments in the Leduån Bridge. The results from the calculations with higher traffic loads are presented in section 14.3.3.
14.3.2 H-piles

If the H-piles are made of steel with yield strength of 440 MPa (S440J2H), then no plastic hinges would be developed according to the original soil model. The worst situation is the alternative soil model and a corrosion depth of 2.4 mm. This situation gives strains in the outermost part of the flanges which exceeds the yield strain (12%).

If the piles instead are made of steel with yield strength of 355 MPa (S355J2G4), then there would be plastic deformations of both corroded and unaffected piles, according to the alternative soil model. The original soil model gives no plastic deformations. The maximum strain range is $1.39\cdot\varepsilon_y$ for piles that have corroded to a depth of 2.4 mm and the alternative soil model. This is about 70% of the variation, $2\cdot\varepsilon_y$, which is needed to achieve annual cycles involving plastic deformations.

Low-cycle fatigue failure is not expected, no matter which soil model that is used in the calculations, which steel that is chosen, and if the piles are corroded or not. Other failure modes than low-cycle fatigue failure has not been analysed.

14.3.3 Higher Traffic Fatigue Load

The fatigue load from the traffic shall according to BRO2004 be modelled as a vehicle with four axles and a weight of 66 tonnes. This load is not sufficient to cause plastic strains in the pipe piles, according to the original soil model. A new calculation is done to illustrate the effect of a higher load. The higher load is modelled as a vehicle with three axles and a weight of 75 tonnes, this load is assumed to always coincide with a lane load of 12 kN/m (the same load as the equivalent load Type 1 in BRO2004 21.2211), see Figure 14:5.

![Figure 14:5 Illustrations of the higher loads that have been assumed.](image)

If these loads are combined with the original soil model then the maximum strain in a pile would be as shown in Table 14:1. The same calculations are also done for the alternative soil model and the results are shown in Table 14:2.
Table 14:1 Pile strains due to the higher traffic load, original soil model.

<table>
<thead>
<tr>
<th></th>
<th>No corrosion</th>
<th>Corrosion 2.4 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_{equ,m}$</td>
<td>4.415 m</td>
<td>4.247 m</td>
</tr>
<tr>
<td>$L_{equ,h}$</td>
<td>4.348 m</td>
<td>4.203 m</td>
</tr>
<tr>
<td>$\varepsilon_\text{Max temp}$</td>
<td>0.001841</td>
<td>0.002009</td>
</tr>
<tr>
<td>$\varepsilon_\text{Min temp}$</td>
<td>0.000223</td>
<td>0.000282</td>
</tr>
</tbody>
</table>

Table 14:2 Pile strains due to the higher traffic load, alternative soil model.

<table>
<thead>
<tr>
<th></th>
<th>No corrosion</th>
<th>Corrosion 2.4 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_{equ,m}$</td>
<td>3.348 m</td>
<td>3.143 m</td>
</tr>
<tr>
<td>$L_{equ,h}$</td>
<td>2.945 m</td>
<td>2.773 m</td>
</tr>
<tr>
<td>$\varepsilon_\text{Max temp}$</td>
<td>0.002824</td>
<td>0.003125</td>
</tr>
<tr>
<td>$\varepsilon_\text{Min temp}$</td>
<td>0.000090</td>
<td>0.000128</td>
</tr>
</tbody>
</table>

**Original soil model**
The pile strains would not exceed the yield strain of 0.0021, and no plastic deformations would be expected even in this rather extreme load situation, which can be described as following.

- +31°C shade air temperature (EBT +34.4°C)
- 18°C temperature difference between deck and girders (girders are warmer). This situation occurs statistically once in fifty years and probably never in the summer.
- A vehicle with a weight of 75 tonnes, 3 axles, and 7.5 m between the first and last axle are crossing the bridge at the same time as 48 tonnes are uniformly distributed in the same lane. Every single axle on the vehicle has an overload of 80% and the gross weight limit is more than two times higher than the allowable weight for a vehicle with 7.5 m between the first and last axle. This type of vehicle will hopefully never be on the roads, and if there are such vehicles on the roads they will probably be very rare.
- The positive effect of the passive soil pressure is not included.

**Alternative soil model**
The more conservative soil model gives compressive pile strains which exceed the yield strain with 50%, for corroded piles. There are no tensile stresses which exceed the yield strength. The highest tensile stress at the opposite side of the pile is about 82% of the yield stress. There will be no reversible cycles with amplitudes of $2\varepsilon_y$. The cycles with the largest amplitudes will be about 72% of $2\varepsilon_y$.

**Conclusion**
A low-cycle fatigue failure of the piles, in the Leduån Bridge, is not expected. There are no indications that reversible plastic strain cycles would occur.
14.4 **Not Included Parameters**

Some parameters that actually would affect the piles strains are not taken into consideration when the pile strains are calculated. These parameters are described in this section, and there are also explanations of why they are not included in the calculations.

14.4.1 **Moments Caused by Dead Load from the Abutments**

The wingwalls and the lower part of the backwalls are cast at the same time and become one structural unit. This unit has its centre of gravity located behind the back wall and will add an extra moment to the piles. This moment will give a negative rotation angle at the top of the piles, and add some extra strains to the pile when the lateral movements are negative (in wintertime) or give some relief of the strains when the lateral movements are positive (in summertime). The Leduån Bridge abutment is illustrated in Figure 14:6. Only the weights from the parts of the abutments that are cast before the rigid connection to the superstructure is formed have been taken into consideration when the centre of gravity was calculated. The weight from the upper part of the backwall and the edge beams, which are cast later, were not taken into the calculations.

![Figure 14:6 Illustration of the Leduån Bridge abutment.](image)

The moment, due to the dead load from the abutment, could be used to pre-stress the piles and would neutralize at least a part of the rotations caused by loads acting on the superstructure. Yet, these rotations are not taken into consideration. Mainly because of the fact that it will not affect a possible fatigue failure since the load is constant. But also since an easy way to calculate how large the real moments in the piles would be, has not been found. It would be dependent of how much of the load that is transferred through the $1.65m^2$ large bottom surface of the abutment, and how large settlements that are expected beneath this surface. A simple model could be based on the assumption that all forces and moment are transferred through the piles. But, as shown in the previous strain calculations, the strains in the piles are likely to be highest in the summer, and the rotations due to the abutment dead load would lower the strains in the summer. An assumption that all of the dead load is transferred through the piles could therefore result in an underestimation of the rotations and an overestimation of the normal forces, in the summer. By not taking this rotation into consideration, the pile rotations in the wintertime might be underestimated. Yet, since pile strains in the winters appear to be rather small compared to the strains in the summer this would not be that important.
15 Fatigue Simulation Model

The simulation of fatigue in integral abutment piles is performed according to the formulas and calculation models described earlier in this report. The given values in design codes for traffic load, temperatures and temperature gradients are replaced by simulations of the varying parameters. The temperature simulation models have been described earlier in Chapter 10 and 11, and the traffic simulation model is described in the following section. The traffic loads from the codes are in the simulation replaced by a traffic load model based on real gross weight measurements in order to get a better estimation of the fatigue.

15.1 Traffic Load Model

The traffic load model in this report is based on truck loads from Bridge Weigh In Motion (BWIM) measurements and AADT (Annual Average Daily Traffic) values measured with the Metor 2000 system. Second hand information has been used, and most of the information has been collected from the report *BWIM-mätningar 2002 och 2003 Slutrapport* (Vägverket 2004). A doctoral thesis, *Traffic Load Effects on Bridges*, written by Getachew (2003) has also been frequently used as a source of information.

**AADT measurements**

The Swedish National Road Administration uses a system called Metor 2000 to collect traffic data from the Swedish roads. This system is rather simple but effective. It consists of two rubber tubes that are placed perpendicular across the road. A pulse is sent to the recording device each time an axel of a vehicle crosses a tube. The pulses from the two tubes are analysed by the Metor system and vehicles are classified in 15 categories depending on the number of axels and axel distances. The vehicle classes according to Metor 2000 are shown in Appendix K.

**BWIM measurements**

Vehicles with a gross weight larger than 3.5 tonnes are all classified as trucks in the results from the BWIM monitoring. The data received from the monitoring gives a lot of information of the vehicles, such as axel loads, number of axels and axel distances. Each vehicle that crosses the bridge can be sorted into a certain vehicle class. There are several different ways of classifying a vehicle.

The BWIM monitoring gives more information than the Metor system, since it also registers the axel weights. If raw data are available from a BWIM monitoring with a satisfying quality, then it would not be necessary to use the Metor system. Yet, since no raw data has been available in this report and second hand information have been used instead, the results from both Metor and BWIM monitoring have been combined to form a traffic load model.

15.1.1 Truck Weight Models

BWIM measurements performed by the Swedish National Road Administration are presented in the report *BWIM-mätningar 2002 och 2003 Slutrapport* (Vägverket 2004). Truck weight distributions are presented from some of the measurement locations. In this report, two of these locations are chosen in order to get two different types of traffic load models. The first one is measurements made at a bridge along the road E22 close to the Swedish town Strängnäs. This location is chosen since it has the highest AADT of the available...
measurement series. The second location is a bridge along the National Road 67 near to Tillberga. The results from the measurements on this bridge showed that a lot of the trucks that crosses the bridge are heavily loaded. This bridge is chosen to be able to study how different truck weight distributions are affecting the fatigue of the piles.

The truck weight distributions that are available have intervals of 5 tonnes. For instance, all vehicles between 5 and 10 tonnes are placed in the same bin in the histogram, and there is no information of how the weights are distributed within that interval. An assumption has been made saying that the weight is uniformly distributed within an interval of 5 tonnes. A more conservative assumption would be that all vehicles within a weight interval have the highest weight in that interval. A weight model made on this assumption would probably overestimate the fatigue, since it would give a maximum load that is frequently occurring. The fatigue is expected to be mainly caused by the heavy vehicles, and the trend in the tail of the distributions is a decreasing frequency as the vehicle weight increases. The assumption that the weight is uniformly distributed within an interval is probably also relatively conservative for the highest and the most interesting weight intervals.

The BWIM measurements performed by Vågverket have only been going on for a week at each location, and there have been some interruptions in the measurements. Therefore, AADT values measured by the Metor system are used instead of the AADT measured with the BWIM-technique, see Table 15:1. It is also assumed that traffic intensity is the same in both directions at the studied road, 50% of the vehicles travels in one lane and 50% in the other.

<table>
<thead>
<tr>
<th>Road</th>
<th>Location</th>
<th>Year</th>
<th>All vehicles</th>
<th>Trucks</th>
</tr>
</thead>
<tbody>
<tr>
<td>E22</td>
<td>Strängnas</td>
<td>2002</td>
<td>13520</td>
<td>1560</td>
</tr>
<tr>
<td>National Road 67</td>
<td>Tillberga</td>
<td>2001</td>
<td>8970</td>
<td>1200</td>
</tr>
</tbody>
</table>

15.1.1.1 E22 - Strängnas
Traffic loads acting on a bridge at the road E22 have been monitored continuously with BWIM-technique by Vågverket, during the time period 2002-09-09 – 2002-09-15. The number of trucks (>3.5 tonnes) that crossed the bridge during this period was 9434, and the mean value of the trucks gross weight was 24.4 tonnes (Vågverket 2004). The distribution of the truck gross weight is presented in Figure 15:1.

![Figure 15:1 Truck gross weight distribution at E22 – Strängnas.](image-url)
15.1.1.2 National Road 67 - Tillberga

BWIM measurements at National Road 67 (Riksväg 67) have been performed by Vägverket during the time period 2002-08-19 – 2002-08-25. The number of trucks (>3.5 tonnes) that crossed the bridge during this period was 4643, and the mean value of the trucks gross weight was 33.1 tonnes (Vägverket 2004). Figure 15:2 shows the truck gross weight distribution.

[Figure 15:2 Truck gross weight distribution at National Road 67 – Tillberga.]

15.1.2 Vehicle Classification

If raw data from BWIM-measurements were available, it would be possible to divide the trucks into several different classes depending on axel load, number of axels and axel distances. Yet, since no raw data have been available the number of vehicle classes is simplified. Getachew (2003) based his Monte Carlo simulation of traffic loads on WIM measurements from road E6. He divided the vehicles into three classes.

- **Group 1:** Passenger cars and vans.
- **Group 2:** Rigid lorries with two to four axels, including busses.
- **Group 3:** Articulated lorries and lorries with trailer.

If the load is doubled then the fatigue increases eight times. The weight of a passenger car is only a few percentage of the weight of a heavily loaded truck. The loads from single passenger cars (Group 1 vehicles) are neglected in this report, since their contribution to the fatigue of the piles are insignificant. The loads from passenger cars are only taken into account when there are traffic queues at the bridge.

The vehicles in Group 2 and 3 span over a weight interval from 3.5 to 75 tonnes, and contain vehicles with different number of axels and different axel distances. The number of different trucks is in this report simplified to four types, since there are no data available for every single truck that has crossed the bridge during the measurement period. The four types of trucks are defined in the following pages.

The Swedish traffic regulation, Trafikförordningen (1998), defines in Chapter 4 12 § and Appendix 1 the maximum allowable loads on public roads in Sweden. These paragraphs have been used as a support in the creation of a traffic model. The paragraphs are summarised by Table 15:2 and Table 15:3.
Table 15:2 Allowable gross weights according to the Swedish Traffic Regulation, Trafikförordningen (1998).

<table>
<thead>
<tr>
<th>Motor driven vehicles</th>
<th>Maximum weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vehicles with two axels</td>
<td>18 ton</td>
</tr>
<tr>
<td>Vehicles with three axels</td>
<td>25 ton</td>
</tr>
<tr>
<td>Vehicles with three axels + extra requirements on suspension and double wheels, see Trafikförordningen (1998)</td>
<td>26 ton</td>
</tr>
<tr>
<td>Articulated bus with three axels</td>
<td>28 ton</td>
</tr>
<tr>
<td>Vehicles with four axels or more</td>
<td>31 ton</td>
</tr>
<tr>
<td>Vehicles with four axels or more + extra requirements on suspension and double wheels, see Trafikförordningen (1998)</td>
<td>32 ton</td>
</tr>
</tbody>
</table>

Table 15:3 Allowable loads according to the Swedish Traffic Regulation, Trafikförordningen (1998).

<table>
<thead>
<tr>
<th>Axel load</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non driving axle</td>
<td>10 ton</td>
</tr>
<tr>
<td>Driving axle</td>
<td>11.5 ton</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bogie load</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axle distance less than 1.0 m</td>
<td>11.5 ton</td>
</tr>
<tr>
<td>Axle distance within 1.0 – 1.3 m</td>
<td>16 ton</td>
</tr>
<tr>
<td>Axle distance within 1.3 – 1.8 m</td>
<td>18 ton</td>
</tr>
<tr>
<td>Axle distance within 1.3 – 1.8 m + extra requirements on suspension and double wheels, see Trafikförordningen (1998)</td>
<td>19 ton</td>
</tr>
<tr>
<td>Axle distance larger than 1.8 m</td>
<td>20 ton</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Triple axle load</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axle distance between the outer axles is less than 2.6 m</td>
<td>21 ton</td>
</tr>
<tr>
<td>Axle distance between the outer axles is larger than 2.6 m</td>
<td>24 ton</td>
</tr>
</tbody>
</table>

The allowable gross weights for articulated lorries and trailers are given in Appendix M in proportion to the distance between the first and last axle.

Getachew (2003) used the distributions in Figure 15:3 when he simulated the traffic loads from vehicle Group 2 and 3. The two figures are based on WIM measurements from road E6 in Sweden, near to Torp.

![Figure 15:3](image-url) Vehicle gross weight distribution in Group 2 and 3, redraw from Getachew (2003).
Most of the vehicles in Group 2, 99% (2.3 standard deviations), have a weight below 20 tonnes, but there are some exceptions with weights up to 28 tonnes. As shown in Figure 15:3, a lot of the vehicles in Group 3 have a weight below 28 tonnes. In this report, all vehicles with a weight less than 28 tonnes are sorted in as Type 1 vehicles. That boundary gives a rather conservative approach to the traffic load, since more than 50% of the Group 3 vehicles, in Getachew’s model, will be modelled as Type 1 vehicles in this report, with at least one axel less.

Overloaded trucks or trucks with at least one overloaded axel occur rather frequently on the Swedish roads. Vägverket (2005) states that 13.8% of the trucks, weighted by the BWIM-technique during 2005, were overloaded. Vehicles with high gross weight, more than 35 tonnes, were overrepresented in the group of overloaded trucks. Every third vehicle above 35 tonnes was overloaded at least on one axel. This is taken into consideration when the model of the trucks is created by allowing some overload. The higher load that is allowable at the driving axel, according to the Swedish Traffic Regulation, is not taken into consideration in the model. All single axels are treated equally.

15.1.2.1 Vehicle Type 1
This truck with three axels will represent the vehicles that Getachew (2003) defined as Group 2 vehicles. The axel distances and the distribution of the gross weight, \( W \), between the axels are the same as for the Equivalent Load Type 1 in BRO2004, see Figure 15:4. The maximum allowable gross weight is set as 28 tonnes, giving a possible gross weight overload of 12.5% compared to the allowable weight of a three axel vehicle.

15.1.2.2 Vehicle Type 2
The Type 2 vehicle is modelled with four axels. The axel distances and the distribution of the gross weight between the axels are the same as for the fatigue load model in BRO2004, see Figure 15:5. The maximum gross weight is set as 38 tonnes, giving a possible gross weight overload of 19% compared to the allowable weight of a four axel vehicle.
15.1.2.3 Vehicle Type 3
The Type 3 vehicle is defined as a vehicle with five axles, shown in Figure 15:6. The maximum gross weight it set as 55 tonnes. That gives a possible gross weight overload of 6%, compared to the maximum gross weight that is allowed for a vehicle with a maximum axel distance of 14.5 m. The possible overload is assumed to be distributed over all axels except the first one, see Figure 15:6. This distribution gives axel loads that can be more than 20% higher than the allowable values.

![Vehicle load model for vehicle Type 3.](image)

15.1.2.4 Vehicle Type 4
The Type 4 vehicle is defined as a vehicle with seven axels, see Figure 15:7. The maximum gross weight it set to 75 tonnes, which is the largest possible gross weight in the simulation. That gives a possible gross weight overload of 25% compared to the maximum gross weight that is allowable for a vehicle with a maximum axel distance of 20.5 m. The possible overload is assumed to be distributed over all axels except the first one, see Figure 15:7. This distribution gives axel loads and bogie loads that can be more than 20% higher than the allowable values.

![Vehicle load model for vehicle Type 4.](image)

15.1.3 Queue weight
The strains caused in the piles by single passenger cars are negligible, since the strain cycles are so small that the number of cycles to failure, \( N_f \), goes towards infinity. Yet, if there are traffic stockings or something else causing queues over the bridge, then it could be important to study the load from all vehicles, not just only trucks.

Getachew (2003) calculated queue weight distributions from WIM measurements performed by Vägverket. Figure 15:8 illustrates how the queue weight is distributed in a 250 m long queue, according to Getachew’s calculations. This distribution is based on unfiltered WIM data, with no consideration taken to the fact that the proportion of trucks is higher during the night than during the day. In his further calculations, Getachew uses a more sophisticated model with periodic variations in the queue weight. In this report, the queue weights are modelled after the simple distribution showed in Figure 15:8, with some modifications.
The probability distribution of the Queue weight has a similar shape as a log-normal distribution. In the simulations, the distribution in Figure 15:8 is replaced by a log-normal distribution which has been adjusted to fit the distribution based on WIM-measurements. Figure 15:9 shows a simulation result from the log-normal distribution and its input parameters.

There are no statistics available of how often a queue occurs and how often a truck is a part of a queue. The most conservative assumption would be that there is always a queue when a truck is crossing the bridge. That assumption is of course not true and would overestimate the fatigue a lot, but it is not that easy to estimate how often a queue occurs at a single road without information about how the traffic intensity is varying over a day.

In the Swedish Bridge Code, no lane load is used in the fatigue calculations. The fatigue load is modelled with a single truck. One alternative would be to neglect the queue weight from the simulations of the varying pile strains. Yet, it will occur and it is possible that the highest pile strains would occur during a traffic queue. The queue weight is therefore brought into the model even if the model is not verified against the reality. In order to not overestimate the fatigue caused by the queue weight too much, the simulated number of queues per day is kept rather low.

An assumption is made, saying that a queue occurs in average ten times a day at the same time as a truck crosses the bridge. When this happens a uniformly distributed queue weight are added to the same lane as the truck load. This assumption is not verified against the
reality, it is only a result of the discussion in the following sentences. The queue weight model is developed assuming 2 m distances between the vehicles. The vehicles have to be almost standing still to keep those small distances. In a normal case with vehicles travelling in 70-110 km/h the distances between the vehicles are much larger. If there is a truck at the middle of a 40 m long bridge travelling in 90 km/h, then the probability is rather low that one or more vehicles are on the bridge in the same lane at the same time. Therefore, the queue weight is only taken into consideration when the traffic is almost standing still. This is assumed to happen in average 10 times a day. That would be a rather high value if the bridge is located on a road with a speed limit of 90 km/h, but it could also be low if the bridge are in the middle of a city or close to a crossing.

If a bridge is long, it might be better to divide the lanes into sectors and generate a queue weight value for each sector. The bridge over Leduvå River has a span length of 40 m and is relatively short, and one lane is in this case treated as only one sector.

### 15.1.4 Summary of the Traffic Load Models

The traffic load models are based on truck weights measured with BWIM-technique at two different locations, E22 Strängnäs and National Road 67 Tillberga. The trucks are divided in four groups depending on their gross weight, vehicle Type 1 - 4. When there are traffic queues over the bridge, the truck weights are combined with a queue weight which is uniformly distributed over the lane in which the truck is. The queues involving a truck are assumed to occur in average 10 times a day. This parameter can of course be varied in the simulations. The queue weight model is a simplified version of a model created by Getachew (2003). The traffic load model works according to the following schedule, see Figure 15:10.

- Truck weight, $W$, is generated.
- $W \Rightarrow$ Vehicle type 1, 2, 3 or 4.
- Queue weight, $q$, is generated
- Normal force is calculated in the most exposed pile.
- Rotation due to the traffic load is calculated
- Pile strain caused by the traffic load is calculated.

![Figure 15:10](image)

*Schematic illustration of the traffic load models.*
The six steps in the generation of strain cycles, due to the traffic loads, should be repeated every time a truck crosses the bridge. The two roads that are used as examples in this study, E22 and National Road 67, have in average 1560 respectively 1200 trucks that are crossing the bridges daily. This would give 68 respectively 53 million traffic load cycles during the 120 year long service lifetime of the bridge. If every individual strain cycle is calculated, the amount of data would be enormous. Therefore, 100,000 traffic load cycles are generated for each road. These load cycles are taken as representative for the whole lifetime, and are repeated until the demanded number of cycles are achieved.

The traffic load simulation is done with the Monte Carlo simulation program Anthill Lite. The input files are presented in Appendix L.

### 15.2 Fatigue Calculations

The fatigue calculations are strain based since plastic deformations are allowed to occur. The number of cycles until failure, \( N_f \), for a certain strain cycle can be estimated according to Coffin-Manson’s universal slope equation or the extrapolated \( \varepsilon \)-\( N_f \) curves. As shown in Chapter 6, the extrapolated \( \varepsilon \)-\( N_f \) curves are more conservative than Coffin-Manson’s model. The first choice in the calculations of number of cycles until fatigue failure is the extrapolated \( \varepsilon \)-\( N_f \) curves. Since it is a more conservative model, simpler to use, and it is the most common used fatigue model in the codes. The S-\( N_f \) curves in BSK99 are defined for \( N_f > 10^3 \), and are extrapolated to 1 cycle. The Coffin-Manson model will not be used as long as \( N_f > 10^3 \).

The extrapolated \( \varepsilon \)-\( N_f \) curves are based on high-cycle fatigue tests, and are not developed for low-cycle fatigue. The Coffin-Manson model is developed for low-cycle fatigue, and will probably give better estimations of the number of cycles until fatigue when there are a lot of cycles giving plastic deformations. A drawback with the Coffin-Manson model is the rather complex equation, see (15.1).

\[
\Delta \varepsilon = 3.5 \frac{\sigma_0}{E} (N_f)^{0.12} + \varepsilon_f^{0.6} (N_f)^{0.6} . \tag{15.1}
\]

Since there are millions of strain cycles that shall be analysed it would be really good with an expression where \( N_f \) is a function of the strain range, \( \Delta \varepsilon \). It seems hard to transform (15.1) to such a function. It is possible to perform a graphical solution of the equation, use an approximation, or use a computer program to solve the equation for each cycle. A graphical solution is time consuming, and an external computer program is not an alternative since the calculation process is automated in Excel.

Chapter 8:71 in BSK99 says that corrosion protection can be a certain fraction of the cross-section area that is allowed to corrode without loosing the necessary strength of the construction. The piles can perhaps be classified as corrosion protected due to the fact that they are overdimensioned to allow a certain depth of corrosion. A more conservative assumption is made in this report. The piles are treated as constructions that will be affected by corrosion, and the lower level of the fatigue endurance limit in the \( \varepsilon \)-\( N_f \) curves is used in the fatigue calculations.
The contributions to the fatigue from different strain cycles are calculated according to the Palmgren-Miner rule for cumulative fatigue. The number of cycles until failure is not calculated for every single strain cycle. The cycles with amplitudes that are within a certain interval are all assumed to have the highest amplitude in that interval. How wide and many intervals that are used depends on the number of strain cycles. The intervals that have been used in this report are presented in the following sections.

### 15.2.1 Definition of Different Strain Cycles

The pile strains in the calculation model are dependent on EBT, temperature gradient, traffic loads, concrete shrinkage and the weight of the construction. Only the first three parameters are variable. *Figure 15:11* shows an example of how pile strains can be varying over a period of three years.

![Total strain variation during three years, traffic loads excluded.](image)

The total strain in the piles is a function of variables with frequencies that spans from seconds up to a year. To be able to analyse the fatigue with the available computer programs, the total pile strain must be separated into different cycles. Two groups of cycles are identified.

- **Temperature cycles**  
  EBT, daily temperature variations, temperature gradient variations, and constant loads (dead loads and shrinkage)

- **Traffic cycles**  
  loads from vehicles

#### 15.2.1.1 Temperature Cycles

The temperature cycles are based on annual temperature cycles on which daily temperature variations are superposed.

**Annual strain cycle**

The annual strain cycle has a period of one year, and an amplitude that mainly are a function of the EBT. The service lifetime of the bridge is 120 years, and the piles must be able to withstand 120 annual strain cycles. Amplitude differences between the annual strain cycles, from one year to another, are expected to be rather small compared to the total amplitude.

The constant load from the superstructure is taken into consideration when the annual strain cycles are calculated. It will not affect the amplitude, but they will add some rotations and normal forces that shift the position of the strain curves in a way that magnifies the strains during the summer. The largest constant rotation is caused by the shrinkage in the concrete deck.
Figure 15:12 shows an example of how the annual strain cycle could vary during three years.

**Figure 15:12** Yearly strain cycle caused by seasonal temperature variations.

**Daily strain cycles**
The daily variations in EBT and temperature gradients will give daily strain cycles with a period of 12-24 hours. The amplitude can vary a lot from one cycle to another since individual values are generated every day during 60 years for the daily maximum and minimum EBT, and the temperature gradients. The strain cycles from these 60 years are taken as representative for the whole lifetime and are therefore assumed to be repeated another time during the bridge service lifetime of 120 years. Figure 15:13 shows an example of a result from a simulation of daily strain cycles during three years.

**Figure 15:13** Daily strain cycles caused by daily temperature variations and temperature gradients.

### 15.2.1.2 Traffic Cycles
The strain cycles in the piles induced by traffic loads will have periods that could be measured in seconds or parts of a second. The number of cycles that occur during a day is modelled by the AADT value for the specific road and location. Figure 15:14 illustrates the simulated pile strains caused by the traffic during three hours. The E22 traffic model has been used with an AADT value of 1560 trucks per day.

**Figure 15:14** Strain cycles due to traffic loads, during three hours.
15.2.2 Counting Cycles
The amplitudes of daily strain cycles are varying a lot in contrast to the annual cycles, which are almost the same from one year to another. The temperature cycles are analysed together, which means that the daily variations have been superposed on the annual cycles. The simple ranges between peaks and valleys are known from the simulations, but it is harder to identify the cycles among these variations. A number of different techniques can be used to identify cycles for irregular loading. *ASM Handbook in Fatigue and Fracture*, states that an agreement appears to have been reached that the preferable method is the rain-flow method. The rain-flow counting technique has therefore been used to identify the temperature cycles in this report.

The strain cycles caused by the traffic loads are counted separately, since the period of such a cycle is very short compared to the temperature cycles. There is no need of any cycle counting technique either, since every single traffic load is modelled as one complete cycle with loading and unloading. If the traffic cycles should be superposed on the temperature cycles it would be necessary to use a time scale in seconds. This would give enormous amounts of data, since it would be necessary to define the temperature at every single traffic load. It would be necessary to define about 1500 temperature values daily. This can be compared to the 2 daily values which now are used in the temperature model. The Monte Carlo simulation program that has been used should only be able to simulate the strain during one month if the traffic cycles were superposed, due to the limitation of 50000 simulation steps. It would certainly be possible to analyse all cycles superposed on each other, but it would demand a computer program which can deal with at least 1 million simulation steps to be able to analyse one year in one simulation.

The traffic loads will however give a contribution to the maximum strains that occurs daily. In order to not underestimate the maximum amplitudes of the daily cycles, a traffic strain is added to the daily maximum strain. The minimum values will not be affected since the traffic always gives positive rotations and translations. *Figure 15:15* illustrates how the temperature strain cycles are combined with the traffic strain cycles.

![Figure 15:15](image-url) Illustration of how daily strain ranges are modelled.
In the upper left corner, the varying strain due to daily temperature changes are illustrated during seven days. In the upper right corner, the varying traffic strain cycles during 12 hour is illustrated. The traffic strains during 12 hours are superposed on a part of the temperature strain curve, in order to illustrate the contribution from the traffic loads. The daily maximum strain will be higher and the minimum strain will not be affected. Daily temperature strain cycles are therefore modelled with a traffic load added on top of the daily maximum values. The strains due to the traffic loads are then analysed separately from the temperature strains. This will not give any extra cycles compared to a model with all cycles superposed on each other, analysed by any cycle counting technique. The number of cycles and the size of the amplitudes will be the same, since the traffic cycles are fully reversible and have periods which are just a fraction of the period of the daily variations. The mean value of the traffic cycles will be irrelevant, and the amplitude will be the only interesting parameter.

The strain that is added on top of the daily maximum temperature strain is set as the maximum strain according to the traffic models. It means the strain caused by a truck with a gross weight of 70 tonnes in the E22 model, and 75 tonnes in the NR67 model.

### 15.2.2.1 Rain-flow Method

A number of different techniques can bee used to identify cycles for irregular loading. The rain-flow method has been used in this report since it seemed to be the most common accepted method.

In the rain-flow method, a cycle is defined as a peak-valley-peak or a valley-peak-valley sequence A-B-C with a second range B-C that exceeds the first range A-B, see Figure 15:16.

![Figure 15:16 Definition of cycles in the rain-flow method, after ASM Handbook.](image)

When a cycle is identified, the range and the mean value are registered and the cycle is removed from the loading sequence. If an A-B-C sequence is identified as no cycle, then the next sequence are checked for a cycle. This process is repeated through the whole loading and unloading history. Figure 15:17, from ASM Handbook (1996) is used to illustrate the counting technique. When the identification of cycles is completed, every single peak and valley will be a part of one and just one cycle. Most of the cycles will in this certain case have periods of about a day, but there will also be cycles with periods up to years.
Since there will be ten thousands of temperature cycles, a computer program is used to perform the rain-flow counting method. A Matlab-script, made by Adam Nieslony (2003), has been used to identify the strain cycles, count them and sort them into bins. The number of cycles until failure will not be calculated for each individual cycle, since there are about 43 800 temperature cycles during the bridge service lifetime. The temperature strain cycles are instead sorted according to their amplitude, and divided in 40 intervals. When the cumulative fatigue is calculated, all cycles within an interval are assumed to have the highest amplitude in that interval. Figure 15:18 illustrates a result from rain-flow counting of simulated temperature strain cycles during 60 years. The sixty annual cycles can be seen in Figure 15:18a and Figure 15:18b as a small separate group with higher amplitudes than the other cycles.

**Figure 15:17** Illustration of the cycle counting technique in the rain-flow method, after ASM Handbook (1996).

**Figure 15:18** (a) Amplitude distribution of temperature induced strain cycles. (b) 3D-model of the distribution of both amplitudes and mean values for temperature induced strain cycles.
16 Fatigue Simulation Results

The results that are presented in this chapter are the calculated cumulative fatigue during 120 years for the outermost piles, which will suffer the highest strain variations. All calculations are done both for unaffected piles and piles that have corroded to a depth of 2.4 mm. The parameters that will be varied are

- equivalent cantilever length (different soil models)
- bridge length
- location
- pile cross-section

The cumulative fatigue failure criterion can be expressed as

$$\sum_{i=1}^{n} \frac{n_i}{N_{f_i}} < 1.$$  \hspace{1cm} (16.1)

16.1 Different Soil Models

The calculations are done for the original soil model, based on Abendroth and Greimann’s model, which assume that the soil stiffness in the pre-drilled holes is zero. The calculations are also done according to the alternative model which assumes that the loose sand in the pre-drilled holes have a stiffness according to \textit{BRO2004}.

16.1.1 Original Soil Model

<table>
<thead>
<tr>
<th>Cumulative fatigue $\sum_{i=1}^{n} \frac{n_i}{N_{f_i}}$</th>
<th>Extrapolated $\varepsilon - N_f$ curves</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Traffic model</strong></td>
<td><strong>E22</strong></td>
</tr>
<tr>
<td>Temp. strain cycles</td>
<td>0.0252</td>
</tr>
<tr>
<td>Traffic strain cycles</td>
<td>0.0487</td>
</tr>
<tr>
<td></td>
<td><strong>0.074</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cumulative fatigue $\sum_{i=1}^{n} \frac{n_i}{N_{f_i}}$</th>
<th>Extrapolated $\varepsilon - N_f$ curves</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Traffic model</strong></td>
<td><strong>E22</strong></td>
</tr>
<tr>
<td>Temp. strain cycles</td>
<td>0.0323</td>
</tr>
<tr>
<td>Traffic strain cycles</td>
<td>0.1244</td>
</tr>
<tr>
<td></td>
<td><strong>0.157</strong></td>
</tr>
</tbody>
</table>
16.1.2 Alternative Soil Model

Table 16:3 Cumulative fatigue for different traffic models, according to extrapolated $\varepsilon$-$N_f$ curves, alt. soil model.

<table>
<thead>
<tr>
<th>$L_{\text{eq}}$ m</th>
<th>Cumulative fatigue $\sum_{i=1}^{n} \frac{n_i}{N_{f_i}}$</th>
<th>$\varepsilon$-$N_f$ curves</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.348 m</td>
<td>$\text{Traffic model}$</td>
<td>$E22$</td>
</tr>
<tr>
<td></td>
<td>$\text{Temp. strain cycles}$</td>
<td>0.0935</td>
</tr>
<tr>
<td></td>
<td>$\text{Traffic strain cycles}$</td>
<td>0.3121</td>
</tr>
<tr>
<td>2.945 m</td>
<td>$\text{Temp. strain cycles}$</td>
<td>0.406</td>
</tr>
<tr>
<td></td>
<td>$\text{Traffic strain cycles}$</td>
<td>0.406</td>
</tr>
</tbody>
</table>

Table 16:4 Cumulative fatigue for different traffic models, according to extrapolated $\varepsilon$-$N_f$ curves, alt. soil model.

<table>
<thead>
<tr>
<th>$L_{\text{eq}}$ m</th>
<th>Cumulative fatigue $\sum_{i=1}^{n} \frac{n_i}{N_{f_i}}$</th>
<th>$\varepsilon$-$N_f$ curves</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.143 m</td>
<td>$\text{Traffic model}$</td>
<td>$E22$</td>
</tr>
<tr>
<td></td>
<td>$\text{Temp. strain cycles}$</td>
<td>0.1269</td>
</tr>
<tr>
<td></td>
<td>$\text{Traffic strain cycles}$</td>
<td>0.5798</td>
</tr>
<tr>
<td>2.773 m</td>
<td>$\text{Temp. strain cycles}$</td>
<td>0.707</td>
</tr>
<tr>
<td></td>
<td>$\text{Traffic strain cycles}$</td>
<td>0.707</td>
</tr>
</tbody>
</table>

16.2 Longer Bridges

A short analyse is made of how the length of the bridge would affect the cumulative fatigue. The calculations are done for corroded piles and with both the original- and the alternative soil model. It would be time consuming to design new superstructures and supports for the longer bridges, therefore some simplifications have been done. The rotations and pile forces from the traffic are kept constant from the Leduån Bridge calculations. A longer bridge would probably need supports or at least a stiffer superstructure. The bridges are therefore assumed to be designed in a way which gives the same rotations and forces, at the abutments, due to the traffic loads. The effect of uneven settlements between the supports is also neglected. The traffic load model that has been used is the fatigue load according to $\text{BRO2004}$.

Table 16:5 Cumulative fatigue for varying bridge length, according to extrapolated $\varepsilon$-$N_f$ curves, orig. soil model.

<table>
<thead>
<tr>
<th>$L_{\text{eq}}$ m</th>
<th>Cumulative fatigue $\sum_{i=1}^{n} \frac{n_i}{N_{f_i}}$</th>
<th>$\varepsilon$-$N_f$ curves</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.247 m</td>
<td>$\text{Bridge length}$</td>
<td>60 m</td>
</tr>
<tr>
<td></td>
<td>$\text{Temp. strain cycles}$</td>
<td>0.0576</td>
</tr>
<tr>
<td></td>
<td>$\text{Traffic strain cycles}$</td>
<td>0.0482</td>
</tr>
<tr>
<td>4.203 m</td>
<td>$\text{Temp. strain cycles}$</td>
<td>0.106</td>
</tr>
<tr>
<td></td>
<td>$\text{Traffic strain cycles}$</td>
<td>0.106</td>
</tr>
</tbody>
</table>
### Table 16:6 Cumulative fatigue for varying bridge length, according to extrapolated \( \varepsilon - N_f \) curves, alt. soil model.

<table>
<thead>
<tr>
<th>Bridge length</th>
<th>60 m</th>
<th>80 m</th>
<th>100 m</th>
<th>150 m</th>
<th>200 m</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Temp. strain cycles</strong></td>
<td>0.2526</td>
<td>0.4343</td>
<td>0.6967</td>
<td>1.7517</td>
<td>3.5615</td>
</tr>
<tr>
<td><strong>Traffic strain cycles</strong></td>
<td>0.1325</td>
<td>0.1325</td>
<td>0.1325</td>
<td>0.1325</td>
<td>0.1325</td>
</tr>
</tbody>
</table>

### 16.3 Different Locations

The bridge location has been varied in order to get an idea of how large influence the climate has on a 40 m long integral bridge. Calculations have been performed for five locations in Sweden.

### Table 16:7 Cumulative fatigue at different locations, according to extrapolated \( \varepsilon - N_f \) curves, alt. soil model.

<table>
<thead>
<tr>
<th>Bridge length</th>
<th>Karesuando</th>
<th>Kiruna</th>
<th>Umeå</th>
<th>Stockholm</th>
<th>Malmö</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Temp. strain cycles</strong></td>
<td>0.1300</td>
<td>0.1268</td>
<td>0.1263</td>
<td>0.1123</td>
<td>0.1153</td>
</tr>
</tbody>
</table>

### 16.4 Alternative Pile Cross-section

**HEM120**

### Table 16:8 Cumulative fatigue for different traffic models, according to extrapolated \( \varepsilon - N_f \) curves, alt. soil model.

<table>
<thead>
<tr>
<th>Traffic model</th>
<th>E22</th>
<th>R67</th>
<th>BRO2004 fatigue load</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Temp. strain cycles</strong></td>
<td>0.0735</td>
<td>0.0871</td>
<td>0.0754</td>
</tr>
<tr>
<td><strong>Traffic strain cycles</strong></td>
<td>0.1005</td>
<td>0.1181</td>
<td>0.1028</td>
</tr>
</tbody>
</table>

### Table 16:9 Cumulative fatigue for different traffic models, according to extrapolated \( \varepsilon - N_f \) curves, alt. soil model.

<table>
<thead>
<tr>
<th>Traffic model</th>
<th>E22</th>
<th>R67</th>
<th>BRO2004 fatigue load</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Temp. strain cycles</strong></td>
<td>0.0735</td>
<td>0.0871</td>
<td>0.0754</td>
</tr>
<tr>
<td><strong>Traffic strain cycles</strong></td>
<td>0.1005</td>
<td>0.1181</td>
<td>0.1028</td>
</tr>
</tbody>
</table>
17 Conclusion and Discussion

The aim of this thesis was to investigate if, how and when low-cycle fatigue failure is a possible failure mode in piles supporting integral abutment bridges. The conclusion is that low-cycle fatigue does not seem to be a problem at all. At least as long as the length of the bridge does not exceed 100 m, according to the calculation models that have been used. Where the bridge is located, inside of the Swedish borders, is not that important for the fatigue, as long as the bridge is rather short (<100 m). Some pile cross-sections seem to be more suitable than others for integral abutment bridges. No general conclusion about pile cross-sections can be stated, since only two cross-sections have been studied. However, the HEM120 that has been oriented for weak axis bending, seems to be a better choice than RR170x10, considering fatigue. The large influence of the lateral soil stiffness can also be noted, higher soil stiffness decreases the fatigue lifetime considerably. In order to build longer integral bridges, lateral soil stiffness must be controlled in one way or another.

The results from the calculations do not show any sign of problems with low-cycle fatigue, but they instead show that there can be problems with high-cycle fatigue caused by the traffic loads, especially in soils with high lateral stiffness. The traffic load model might be a bit conservative, more about that in the next section Possible Sources of Errors. If we leave out of account the possible errors. The fatigue load, given in BRO2004, seems to underestimate the fatigue compared to the two traffic models. The fatigue load model in BRO2004 consists of a vehicle with a weight of 66 tonnes, and the load is assumed to occur $4 \times 10^5$ times during the bridge service lifetime. The E22 model gives $4.37 \times 10^5$ vehicles with a weight of more than 66 tonnes during the bridge lifetime, and the NR67 model gives $8.20 \times 10^5$ vehicles. The larger number of trucks with a weight of 66 tonnes or more can explain a part of the more conservative fatigue estimations which are achieved. But, since only 50% of the vehicles are assumed to travel in one specific lane, giving the worst load in one specific pile, this would not be the main reason. Yet, the largest contribution to the fatigue comes from allowing vehicle weights heavier than 66 tonnes. A vehicle with a weight of 75 tonnes gives a contribution to the fatigue which in this case is 86% higher than the contribution from a vehicle with a weight of 66 tonnes. And the contribution to the fatigue from a 70 tonnes vehicle is 34% higher than from a 66 tonnes vehicle. These relationships are based on calculations with the original soil model and corroded pipe piles. The passive soil pressure has been assumed to resist abutment rotations less than 1.51‰. However, the fatigue load in the Swedish Bridge Code can not be questioned due to the results in this report. The accuracies of the BWIM measurements, which the calculations are based on, are not known. The measurement periods are very short, one week, and it is possible that these periods are not representative for the annual traffic. The BWIM measurements are second hand information, and it is not known exactly how the measurements have been performed and what type of problems that might have occurred during the measurements.

All assumptions that are made are tried to be conservative, this fact could have lead to an overestimation of a lot of parameters. Rotations of the abutments due to traffic loads are a major reason to the frequently varying stresses/strains in the piles. These rotations are based on an assumption that the connection between superstructure and abutment are monolithic, and 100% rigid. This assumption is not completely true, and the rotations are not entirely transferred to the top of the piles. There will be a loss in the connection between the superstructure and the abutments, and the abutment back walls might also deflect a bit. The
abutments are in this report treated as stub abutments without any deflections. The rotations of the abutments could be reduced by using a superstructure with higher bending stiffness. Yet, the largest profit with integral abutment bridges is the rather cheap construction. A stiffer superstructure means more steel and more costs, and then the profit of the integral abutment bridge might be lost. The bridge that has been analysed is a composite bridge, and it has not been studied whether or not a concrete construction would give smaller abutment rotations.

I have not read anything in the literature about problems with high-cycle fatigue in integral abutment piles. I am therefore a little bit sceptical to the result. The calculation model that has been used might fail to describe the stresses/strains in the piles. Results from a monitoring of pile strains on the existing bridge would be very interesting.

The studied bridge may not be a representative bridge of the roads E22 and NR67, since it is very narrow with 2 lanes and a total width of 5 m. The AADT-value for the road over the Leduån River will probably be just a few percentages of the AADT at the two studied roads. Consequently, the results from the fatigue simulations can not be directly applied to the Leduån Bridge since the fatigue due to the traffic would be much lower. It might have been a better choice to use a bridge with 4-lanes as an example, or at least a wider 2 lane bridge, when the traffic models were developed for roads with high traffic intensity.

17.1 Possible Sources of Errors

The traffic load models were developed to be a bit conservative. The axel loads as well as the vehicle gross weight were allowed to exceed the limits in the Swedish Traffic Regulation. However, the vehicle gross weights have not been exaggerated. They have been taken from BWIM measurements performed by Vägverket. The probability distribution of the gross weight was given in intervals of 5 tonnes. This could lead to an overestimation of the load in the “tail” of the distribution. For instance, it is possible that most of the vehicles in the highest weight interval 70-75 tonnes had a weight that barely exceeded 70 tonnes. But since there was no information available of how the loads were distributed within the 5 tonnes intervals, a uniform distribution was assumed. This might have caused a higher cumulative fatigue than a more exactly distribution of the gross weight would have. The vehicle models are also a possible source of error. Without having the raw data from the BWIM measurements it is hard to model the traffic load in a satisfying way, since a lot of assumptions and simplifications have to be done. The vehicle models that have been used are more an adaptation to the limits, given by the Swedish Traffic Regulations, than models of real vehicles. The axel distances and length of vehicles are not based on any specific vehicles.

The annual temperature models are based on measurements during 60 years, spanning over a period from 1930 to 1990. The temperature model does not take into consideration that we right now might have an increasing mean temperature which could continue to rise in the future. A couple of degrees higher mean temperature would probably not affect the pile fatigue that much. A worse situation would arise if the amplitude of the annual temperature cycles got larger, warmer summers and colder winters. Such a tendency has not been observed as far as I know. The daily temperature model is rather conservative, the daily amplitudes are varying more in the model than in the reality. This is a result of the fact that the model does not take into consideration the former temperatures when it generates a new
one. In the reality the temperature might vary in a small interval during a period of days up to weeks. For instance, it is rather common that the temperature in the wintertime is varying between -20 and -25 °C during a week or two. The probability of getting such small variations in the simulated temperature during a week is very low. The positive and negative temperature gradients have also been assumed to always occur at the part of the day when they would give the most disadvantageous effect. Negative gradients are assumed to occur during the day giving the worst situation. In the reality positive gradients would be expected in the day and negative gradients in the night. The influences from the temperature gradients have probably been exaggerated in this report. But, since their influences are not that big it does not seem to affect the results to much.

17.2 Future Work

It would be interesting if someone created a traffic model based on raw data from BWIM measurements, it might be a subject for a future master thesis. This work should be done in cooperation with Vägverket, since they have experiences from the BWIM technique and the equipment that is necessary. Vägverket have performed a lot of BWIM measurements, and they have certainly an extensive database already. It would probably take a lot of time and effort to transform raw data to a traffic model, based on hundreds of different vehicles and axel distances. It would be really nice with some kind of computer program, which only needed the input from the BWIM measurements to create simulations of the traffic during the bridge lifetime. Then it would be rather easy to create traffic load models for specific roads. In a future it might be possible to design a bridge more after the real traffic loads and intensity, than the values given in codes. It would also be interesting if someone made a full-scale test, and monitored the pile strains in an integral bridge in Sweden. A lot of bridges have been monitored in the USA, but it would be interesting to see if they performed in the same way in our climate and with our traffic loads. Different bridge design codes might also lead to another design and behaviour of the bridges.
Conclusion and Discussion

Low-cycle Fatigue of Steel Piles in Integral Abutment Bridges
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Low-cycle Fatigue of Steel Piles in Integral Abutment Bridges


Appendixes
Low-cycle Fatigue of Steel Piles in Integral Abutment Bridges
Appendix A

Non-linear Temperature Differences in Different Types of Superstructures.

<table>
<thead>
<tr>
<th>Group</th>
<th>Type of construction</th>
<th>Temperature difference (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Positive temperature difference</td>
</tr>
<tr>
<td>1a.</td>
<td>Steel deck on steel box girders</td>
<td>$T_0 = 24°C$</td>
</tr>
<tr>
<td></td>
<td>40mm surfacing</td>
<td>$T = 14°C$, $T_s = 8°C$</td>
</tr>
<tr>
<td>1b.</td>
<td>Steel deck on steel truss or plate girders</td>
<td>$T_0 = 21°C$</td>
</tr>
<tr>
<td></td>
<td>40mm surfacing</td>
<td>$T = 19°C$, $T_s = 11°C$</td>
</tr>
<tr>
<td>2.</td>
<td>Concrete deck on steel box, truss or plate girders</td>
<td>$T_0 = 13°C$</td>
</tr>
<tr>
<td></td>
<td>100mm surfacing</td>
<td>$T = 16°C$, $h_t = 0.6m$</td>
</tr>
<tr>
<td>3.</td>
<td>Concrete slab or concrete deck on concrete beams or box girders</td>
<td>$h_t = 0.3A$ but $\leq 0.15m$</td>
</tr>
<tr>
<td></td>
<td>100mm surfacing</td>
<td>$h_t = 0.2h$ but $\leq 0.25m$</td>
</tr>
</tbody>
</table>

Figure A:1 Non-linear temperature differences in different type of superstructures.
Appendix B

Equivalent Cantilever Length

One way of designing piles is to treat them as beam-columns. Equivalent cantilevers are replacing the piles in design calculations.

Pile capacity

Lateral deformations of integral abutment piles will in general be restricted to the upper part of the piles. The piles can be simplified as cantilever beams, without any transversal loads between the ends. The lower end of the pile is fixed at a certain depth in the soil, \( l_e \), and the upper end could be either fixed or hinged. This report is focused on fully integral abutments, and the only model that is studied is one with fixed pile tops. Figure B:1 shows how an actual pile system can be modelled with an equivalent cantilever, according to a design approach by Abendroth and Greimann (1989, 2005).

![Figure B:1 Actual pile system and the equivalent cantilever model.](image)

The length of the pile in the equivalent cantilever model, \( L_{equ} \), is a function of pile and soil properties, and can be calculated as

\[
L_{equ} = l_e + l_u, \tag{B.1}
\]

where \( l_e \) is the length of the pile from the fixed lower end up to the undisturbed soil surface, and \( l_u \) is the length of the pile above the undisturbed soil surface. Abendroth and Greimann (1989) used the critical length of a pile, \( l_c \), to non-dimensionalize \( l_e \) and \( l_u \). The critical length of a pile, embedded in soil, is the depth beyond which the pile almost behaves as it was infinitely long (Fleming et al. 1992). Lateral movements at the top of the piles will not considerably affect the pile deeper than \( l_c \). No induced shear forces or lateral displacements are assumed to take place below this depth. The lateral displacements and bending moments below \( l_c \), will only be about 4% of those at the top of the pile, according to Abendroth and Greimann (2005). If the lateral soil stiffness, \( k_h \), is constant along the depth, then the critical length of a pile can be calculated as
where \( E_p \) and \( I_p \) are the pile elasticity modulus respectively the pile moment of inertia. The soil stiffness is in general not constant along the depth, and a model with a varying soil stiffness are often more useful. The equivalent uniform lateral soil stiffness parameter, \( k_{eh} \), was introduced by Abendroth and Greimann (1989, 2005), in order to transform varying soil stiffness into an equivalent uniform soil stiffness value, see Figure B:2. The transformation is done by calculating the external work for a real soil model (varying soil stiffness), and then assuming that it would be equal to the external work in a uniform soil model, see (B.3). The part of a pile that is affected by the lateral displacements, the active length, is assumed to be one-half of the critical length. The lateral displacements are denoted \( y \).

\[
I_c = 4 \cdot \frac{E_p I_p}{k_h}, \quad (B.2)
\]

\[
\int_{l/2}^{l/2} k_h(z) y^2(z) \frac{dz}{2} = \int_{l/2}^{l/2} k_{eh} y^2(z) \frac{dz}{2} \quad (B.3)
\]

Figure B:2 (a) Actual soil stiffness  
(b) Equivalent uniform soil model, according to Abendroth and Greimann (1989)

The equivalent soil stiffness parameter can be calculated by an iterative calculation with the following steps, described by Abendroth and Greimann (1989, 2005).

1. An initial value of \( k_{eh} \) is assumed
2. Calculate the second moment, \( I_k \), of the \( k_h(z) \) diagram, taken about a line at a depth of \( l/2 \).
   \[
   I_k = \int_{l/2}^{l/2} k_h(z) \left( \frac{l}{2} - z \right)^2 \, dz \quad (B.4)
   \]
3. Calculate a new value of \( k_{eh} \) by following expression
   \[
   k_{eh} = \frac{3l}{(l/2)^3} \quad (B.5)
   \]
4. Repeat step 2-3 until the input value of \( k_{eh} \) equals the output value.
The constant lateral soil stiffness value, $k_h$, in (B.2) is substituted by the value of $k_{eh}$, which has been achieved through the iterative calculation. The critical length, $l_c$, can then be calculated according to (B.2).

The critical length is then used in order to calculate the equivalent cantilever length, $L_{equ}$, by using the relationship between its two components, $l_e$ and $l_u$, and the critical length. Abendroth and Greimann (1989) have developed three equations to determine three types of equivalent cantilever lengths. They can be used in three different purposes in the design of an integral abutment bridge. These equations are based on the following three assumptions.

1. The lateral pile stiffness of the equivalent cantilever pile is equal to the lateral stiffness of a pile embedded in soil.

2. The maximum moment in the equivalent cantilever equals the maximum moment in a pile embedded in soil.

3. The elastic buckling load of an equivalent cantilever equals the elastic buckling load of a pile embedded in soil.

Abendroth and Greimann’s three equations for fixed headed piles are plotted in Figure B:3.

![Figure B:3 Equivalent cantilever lengths according to Abendroth and Greimann (1989, 2005)](image)

The equivalent cantilever length based on horizontal stiffness is used to calculate the lateral thermal movement and the moments induced from these. The equivalent length for maximum moment is used to calculate the forces and moments due to gravity loads, and the elastic buckling equivalent length is used to calculate the axial compressive strength of the pile as a structural member.
Appendix C

**Rankine’s Theory of Earth Pressure**

Rankine’s theory of earth pressure, described by Craig (1997), considers the stress state in a soil when the condition of plastic equilibrium has been reached. The resulting force, $F_p$, on an abutment wall due to the passive soil pressure can be calculated as

$$ F_p = \int_0^H p_{\text{pass}} \, dz $$

where $H$ is the height of the abutment wall and $p_{\text{pass}}$ is the passive soil pressure. The passive soil pressure is a measure of a soil’s resistance to lateral compression, and is defined as following

$$ p_{\text{pass}} = K_p \cdot \gamma \cdot z + 2 \cdot c \cdot \sqrt{K_p} $$

where $K_p$ is the passive pressure coefficient, and $\gamma$ is the unit weight of the soil material. The second term is dependent on the cohesion, $c$, of the soil material. In general, well drained granular soil with no cohesive strength is used as backfill behind an abutment wall. The second term in (C.2) can therefore be neglected in an analysis. For granular soils, (C.1) can be written as

$$ F_p = \frac{K_p \cdot \gamma \cdot H^2}{2} $$

The passive pressure coefficient, $K_p$, is defined as

$$ K_p = \frac{1 + \sin \phi}{1 - \sin \phi} $$

where $\phi$ is the soil friction angle. This gives the following expression for the resulting force caused by the passive soil pressure

$$ F_p = \frac{\gamma \cdot H^2}{2} \left[ \frac{1 + \sin \phi}{1 - \sin \phi} \right] $$

where $\phi$ is the soil friction angle. This gives the following expression for the resulting force caused by the passive soil pressure

$$ F_p = \frac{\gamma \cdot H^2}{2} \left[ \frac{1 + \sin \phi}{1 - \sin \phi} \right] $$

where $\phi$ is the soil friction angle.
Appendix D

Ductility Criterion for H-piles – Developed by Abendroth, Greimann, Jones and Ebner

In order to prevent local buckling, due to the lateral movement of the top of the pile, the flange width to thickness ratio must be limited. Abendroth and Greimann (1989) present a ductility criterion based on the condition that the moment-rotation demand do not exceed the capacity. The equations below are valid for fully integral abutment piles.

\[
2\left(\frac{\Delta_{abut}}{L} = \frac{M_p \cdot L}{6EI}\right) + \theta_w \leq \frac{3C_i M_p L}{4EI} \tag{D.1}
\]

The inelastic rotation capacity reduction factor, \(C_i\), is given as

\[
C_i = \frac{19}{6} - \frac{b_f \sqrt{f_y}}{60 t_f}. \tag{D.2}
\]

(D.1) can also be written in terms of lateral displacements of the pile head.

\[
\Delta_{abut} \leq \frac{F_b S L^2}{6EI} \cdot (0.6 + 2.25 \cdot C_i) \tag{D.3}
\]

\(M_p\) plastic moment capacity  
\(\theta_w\) pile head rotation  
\(C_i\) inelastic rotation capacity reduction factor  
\(b_f\) flange width  
\(t_f\) flange thickness  
\(f_y\) yield strength  
\(E\) the elastic modulus  
\(I\) moment of inertia  
\(L\) pile length  
\(F_b\) allowable bending stress  
\(S\) section modulus
Appendix E

Isothermal Maps over Sweden

The maps shown in Figure E:1 are based on shade air temperature measurements from 148 meteorological stations spread over the whole country. The probabilities that the maximum and the minimum temperatures are exceeded once a year are 0.02, which is equivalent with an interval of 50 years. These maps are taken from the Swedish Bridge Code - BRO2004 (Vägverket 2004).

Figure E:1 Isothermal maps of $T_{\text{min}}$ to the left and $T_{\text{max}}$ to the right [$^\circ$C].
Appendix F

Calculations of the Gravity Induced Moments - $M_w$

Abendroth and Greimann (1989) proposed a structural model, according to Figure F:1, in order to perform a simplified structural analysis. The contribution from the gravity loads to the pile moments can be calculated in a conservative way by making some simplifications of the structural system.

![Figure F:1 Idealized structural model, proposed by Abendroth and Greimann (1989)](image)

The bending stiffnesses of bridge superstructures are in general much higher than the bending stiffness of the integral abutment piles, in many cases more than hundred times higher. The stiffness of the superstructure is assumed to be unaffected by the restraint from piles and soil. The continuity of the superstructure at the first support is also neglected, and conservatively simplified to a simply supported beam. These assumptions make it possible to calculate the end rotation, $\theta_w$, of a bridge structure due to the gravity loads. Since the integral abutment piles are rigidly connected to the abutments, they will also rotate by $\theta_w$. This approximation is conservative in many ways. For instance, the joint between the superstructure and the abutments may not be 100% rigid, and the rotation of the pile head would therefore be less than $\theta_w$, which will be an upper limit. The end rotation of the bridge structure, and the upper bound for the pile rotation, can be calculated as

$$\theta_w = \frac{qL_{eqs}^3}{24E_g I_g}, \quad (F.1)$$

where $q$ is the uniformly distributed gravity load, $L_{eqs}$ is the length of the end span, $E_g$ is the elasticity modulus of the bridge girders, and $I_g$ is the moment of inertia for the bridge girders. The rotation angle at the top of the pile can then be used to calculate the moment applied at the top of the piles, $M_w$.

$$M_w = \left( \frac{4EI}{L_{eqs}} \right) \theta_w \quad (F.2)$$
Appendix G

Input Data to Temperature Models

The temperatures that are used in this report are taken from two sources, *Klimatdata för Sverige* (Teasler 1972) and *Temperature and Precipitation in Sweden 1961-90* (Alexandersson et al. 1991).

**Table G:1** Monthly mean temperature [°C], based on temperatures from 30-years 1961-1990

<table>
<thead>
<tr>
<th></th>
<th>Karesuando</th>
<th>Kiruna</th>
<th>Umeå</th>
<th>Stockholm</th>
<th>Malmö</th>
</tr>
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**Table G:2** Monthly mean value of the daily maximum temperatures [°C], based on temperatures from 30-years 1931-1960.

<table>
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<td>5.0</td>
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<td>6.7</td>
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<td>11.9</td>
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<td>3.9</td>
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</table>
Table G:3  Monthly mean value of the daily minimum temperatures [°C], based on temperatures from 30-years 1931-1960

<table>
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<td>-14.6</td>
<td>-7.5</td>
<td>-1.9</td>
<td>-0.1</td>
</tr>
</tbody>
</table>

The following input data have been used to adapt the annual temperature model to measured temperatures.

\[
T_s = T_{s, \text{amp}} \cdot \sin \left( \frac{t_d}{365} \cdot 2\pi - t_0 \right) + T_{s, \text{m}} \quad (G.1)
\]

Table G:4  Input data to adjust the daily mean, max and min temperature model to measured values in the five studied locations.

<table>
<thead>
<tr>
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<tr>
<td></td>
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<td>1.02</td>
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Appendix H

Temperature Models for Different Locations

The average value of the daily mean, maximum, and minimum temperatures for each month measured during 30 year periods, 1931-1960 and 1961-1990, are compared to the sinus temperature model and the polynomial temperature model. All temperatures are given in °C.

Kiruna

![Daily mean temperature in Kiruna](image1)

![Daily maximum temperature in Kiruna](image2)

![Daily minimum temperature in Kiruna](image3)

Karesuando

![Daily mean temperature in Karesuando](image4)

![Daily maximum temperature in Karesuando](image5)

![Daily minimum temperature in Karesuando](image6)
Low-cycle Fatigue of Steel Piles in Integral Abutment Bridges

**Umeå**

**Figure H:7** Daily mean temperature in Umeå

**Figure H:8** Daily maximum temperature in Umeå

**Figure H:9** Daily minimum temperature in Umeå

**Stockholm**

**Figure H:10** Daily mean temperature in Stockholm

**Figure H:11** Daily maximum temperature in Stockholm

**Figure H:12** Daily minimum temperature in Stockholm

**Malmö**

**Figure H:13** Daily mean temperature in Malmö

**Figure H:14** Daily maximum temperature in Malmö

**Figure H:15** Daily minimum temperature in Malmö
Appendix I

Example of Graphical Results from Temperature Simulations

Figure I:1 Graphical results from shade air temperature simulation in Kiruna, 50 years.
Simulated shade air temperature compared with EBT in Kiruna, during 2 years

Figure 1.2 Comparison between shade air temperatures and EBT’s.
Appendix J

Cross-section Calculations

According to the theory of elasticity, the composite action between concrete deck and steel girders can be taken into account by replacing the concrete deck with a rectangular cross section of massive steel. The height of the fictitious cross section is the same as the average height of the concrete deck, \( h_{\text{deck}} \), and the area, \( \Delta A \), is calculated as

\[
\Delta A = A_{\text{deck}} \cdot \frac{E_{\text{deck}}}{E_{\text{girder}} \cdot 1.2 \cdot (1 + \varphi)},
\]

(J.1)

where \( \varphi \) is a shrinkage factor which is 2 for long term loading and 0 for short-term loading. The average height of the bridge deck is 244 mm.

\[
\Delta A_{LT} = 1.50 \cdot \frac{35}{210 \cdot 1.2 \cdot (1 + 2)} = 0.0694 \text{ m}^2
\]

\[ b_{\text{equ}}^{LT} = 0.284 \text{ m} \]

\[
\Delta A_{ST} = 1.50 \cdot \frac{35}{210 \cdot 1.2 \cdot (1 + 0)} = 0.2083 \text{ m}^2
\]

\[ b_{\text{equ}}^{ST} = 0.854 \text{ m} \]

The moment of inertia is calculated for the two types of girders that are used in the bridge. Results from the calculations are shown in Table J:1 and J:2. The e-axis is positive downwards and has its origin in the layer between the upper flange and the deck.

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<th>Table J:1 Cross section properties, long term loading</th>
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<tr>
<td></td>
</tr>
<tr>
<td>Deck</td>
</tr>
<tr>
<td>Upper flange</td>
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<tr>
<td>Web</td>
</tr>
<tr>
<td>Lower flange</td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>

<p>| <strong>end girder</strong>                                      | <strong>Total cross-section</strong> |
|                                                      | <strong>b [mm]</strong> | <strong>t [mm]</strong> | <strong>A [mm²]</strong> | <strong>e [mm]</strong> | <strong>eA [mm³]</strong> | <strong>I [mm⁴]</strong> |
| Deck                                                | 142        | 244        | 34648       | -122       | -4227056     | 1.41E+10    |
| Upper flange                                        | 600        | 25         | 15000       | 13         | 187500       | 3.744E+09   |
| Web                                                 | 1234       | 11         | 13574       | 642        | 8714508      | 1.952E+09   |
| Lower flange                                        | 800        | 45         | 36000       | 1282       | 46134000     | 2.132E+10   |
|                                                      | 99222      | 512        | 50808952    | 4.112E+10  |</p>
<table>
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<tr>
<th></th>
<th>b [mm]</th>
<th>t [mm]</th>
<th>A [mm²]</th>
<th>e [mm]</th>
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<td>104188</td>
<td>-122</td>
<td>-12710936</td>
<td>1.202E+10</td>
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<tr>
<td><strong>Upper flange</strong></td>
<td>500</td>
<td>25</td>
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**Total cross-section**

<p>| | | | | | | |</p>
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<tr>
<td><strong>Deck</strong></td>
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<td>0.0987 m⁴</td>
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<td><strong>Upper flange</strong></td>
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<td><strong>Web</strong></td>
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<td><strong>Lower flange</strong></td>
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<td>0.1158 m⁴</td>
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<tr>
<td><strong>Deck</strong></td>
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<td>0.4617 m³</td>
<td></td>
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<td></td>
</tr>
<tr>
<td><strong>Upper flange</strong></td>
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<td>0.1099 m³</td>
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<tr>
<td><strong>Web</strong></td>
<td></td>
<td>0.1158 m⁴</td>
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<tr>
<td><strong>Lower flange</strong></td>
<td></td>
<td>0.4617 m³</td>
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<td>0.1099 m³</td>
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Table J:2 Cross section properties, short term loading
## Appendix K

### Vehicle Classification System – Metor 2000

<table>
<thead>
<tr>
<th>Vehicle Group MC</th>
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<tr>
<td>Vehicle Class MC</td>
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<td>$80 \text{ cm} &lt; B &lt; 180 \text{ cm}$</td>
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<td>Vehicle Class P20</td>
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<td>$180 \text{ cm} &lt; C &lt; 330 \text{ cm}$</td>
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<td>Vehicle Class P21</td>
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<td>$180 \text{ cm} &lt; C &lt; 330 \text{ cm}$, $330 \text{ cm} &lt; D &lt; 600 \text{ cm}$</td>
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<tr>
<td>Vehicle Class P22</td>
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<td>$330 \text{ cm} &lt; D &lt; 600 \text{ cm}$, $600 \text{ cm} &lt; E &lt; 1050 \text{ cm}$</td>
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<td>180 cm &lt; C &lt; 330 cm</td>
</tr>
<tr>
<td></td>
<td>330 cm &lt; D &lt; 600 cm</td>
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<tr>
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<td><img src="image3" alt="Diagram" /></td>
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<td>180 cm &lt; C &lt; 330 cm</td>
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<tr>
<td></td>
<td>330 cm &lt; D &lt; 600 cm</td>
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<tr>
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<td>180 cm &lt; C &lt; 330 cm</td>
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<tr>
<td></td>
<td>330 cm &lt; D &lt; 600 cm</td>
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<td></td>
<td>180 cm &lt; C &lt; 330 cm</td>
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<td>180 cm &lt; C &lt; 330 cm</td>
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<td></td>
<td>180 cm &lt; C &lt; 330 cm</td>
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<td>330 cm &lt; D &lt; 600 cm</td>
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<tr>
<td></td>
<td>600 cm &lt; E &lt; 1050 cm</td>
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Appendix L

Input Data for Monte Carlo Simulations

Simulation of strain cycles due to traffic loads – pipe piles

Input distributions

<table>
<thead>
<tr>
<th>W</th>
<th>Truck weight probability distribution</th>
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</thead>
<tbody>
<tr>
<td>q</td>
<td>Queue weight probability distribution</td>
</tr>
<tr>
<td>p</td>
<td>Uniform distribution between 0 and 1</td>
</tr>
<tr>
<td>t</td>
<td>Calculation step number</td>
</tr>
</tbody>
</table>

Code

Str = POS(2*b+t+1)*Str1+(1-POS(2*b+t+1))*Str2
b = int(t/2)
str1 = V1(Ap+Le/2)*H*Rrot + (2*d/2*L)*Rrot
str2 = V2(Ap+Le/2)*H*Rrot + (2*d/2*L)*Rrot
Rrot = POS(O+Rsoilmax)*O+Rsoilmax+(1-POS(O+Rsoilmax))*O
O = Z + (q*A1000*L^3)/(24*E*I)
V1 = (N+(q*A1000*L)/2)/6+(N+(q*A1000*L)/2)*1000/12,7
V2 = (N+(q*A1000*L)/2)/6-(N+(q*A1000*L)/2)*1000/12,7
Z = POS(W-m1)*Za+(1-POS(W-m1))*R1
Za = POS(W-m2)*Zb+(1-POS(W-m2))*R2
Zb = POS(W-m3)*R4+(1-POS(W-m3))*R3
Zc = POS(W-m4)*R5+(1-POS(W-m4))*R6
Zd = POS(W-m5)*R6+(1-POS(W-m5))*R7
R1 = (W*10000/L)*(L-a1)*L/6*E*I + (1-(L-a1)^2)/(L^2)
R2 = (W*10000/L)*(L-a2)*L/6*E*I + (1-(L-a2)^2)/(L^2)
R3 = (W*10000/L)*(L-a3)*L/6*E*I + (1-(L-a3)^2)/(L^2)
R4 = (W*10000/L)*(L-a4)*L/6*E*I + (1-(L-a4)^2)/(L^2)
R5 = (W*10000/L)*(L-a5)*L/6*E*I + (1-(L-a5)^2)/(L^2)
R6 = (W*10000/L)*(L-a6)*L/6*E*I + (1-(L-a6)^2)/(L^2)
R7 = (W*10000/L)*(L-a7)*L/6*E*I + (1-(L-a7)^2)/(L^2)

: [-] Pile strain due to traffic loads every
: : every second value is str1 respectively str2
: : [1] Pile strain in the most exposed pile, vehicle in lane 1
: : [1] Pile strain in the most exposed pile, vehicle in lane 2
: : [1] Rotations after reduction due to the backfill soil pressure
: : Sum of the rotations due to the traffic loads
: : Normal force, in the most exposed pile, vehicle in lane 1 [N]
: : Normal force, in the most exposed pile, vehicle in lane 2 [N]
: : If the vehicle weight is larger than m1 THEN the rotation is Za otherwise R1
: : If the vehicle weight is larger than m2 THEN the rotation is Zb otherwise R2
: : If the vehicle weight is larger than m3 THEN the rotation is R4 otherwise R3
: : If the vehicle weight is larger than m1 THEN the vertical force at each abutment is Na otherwise N1
: : If the vehicle weight is larger than m2 THEN the vertical force at each abutment Nb otherwise N2
: : If the vehicle weight is larger than m3 THEN the vertical force at each abutment N4 otherwise N3

: Rotations caused by a Type 1 vehicle with 3-axels
: Rotations caused by a Type 2 vehicle with 4-axels
: Rotations caused by a Type 3 vehicle with 5-axels
: Rotations caused by a Type 4 vehicle with 7-axels

: The vertical force at the abutment with largest rotations due to the Type 1 vehicle. [N]
: The vertical force at the abutment with largest rotations due to the Type 2 vehicle. [N]
: The vertical force at the abutment with largest rotations due to the Type 3 vehicle. [N]
\[ \left( W \times 10000 \times 11/75 \right) \left( L - a_4 - 7.5 \right) / L + \left( W \times 10000 \times 11/75 \right) \left( L - a_4 - 13 \right) / L + \left( W \times 10000 \times 11/75 \right) \left( L - a_4 - 14.5 \right) / L + \left( W \times 10000 \times 9/75 \right) \left( L - a_4 - 20.5 \right) / L \]

: The vertical force at the abutment with largest rotations due to the Type 4 vehicle. [N]

\[ \text{qA} = \text{POS}(p-(s/Tr)) \times 0 + (1-\text{POS}(p-(s/Tr))) \times q \]

: IF \( p \) is larger than \( s/1560 \) THEN the queue weight is zero otherwise \( q \)

\[ d = \text{dypile} - 2 \times \text{corr} \times 0.001 \]

: [m] Pile diameter after corrosion

\[ \text{dypile} = 0.1683 \]

: [m] Pile outer diameter

\[ \text{tpile} = 10 \]

: [mm] Thickness of the pile material

\[ \text{corr} = 0 \]

: [mm] Depth of the corrosion

\[ R_{\text{soil max}} = -0.00151 \]

: [-] Max reduction of the rotations due to the backfill soil pressure.

\[ L = 40 \]

: Bridge length [m]

\[ E = 210 \times 10^9 \]

: Steel E-modulus [Pa]

\[ I = 0.0987 \]

: [m^4] Moment of inertia for the superstructure, short term loading

\[ m_3 = 55 \]

: Weight limit for Type 3 vehicles [ton]

\[ m_2 = 38 \]

: Weight limit for Type 2 vehicles [ton]

\[ m_1 = 28 \]

: Weight limit for Type 1 vehicles [ton]

\[ H = 1.65 \]

: Abutment height [m]

\[ L_{\text{em}} = 4.415 \]

: Equivalent cantilever length moment [m]

\[ L_{\text{eh}} = 4.348 \]

: Equivalent cantilever length lateral stiffness [m]

\[ a_1 = 14.133 \]

: The last axels position on the bridge causing the largest rotations of the abutments, Type 1 vehicles [m]

\[ a_2 = 12.625 \]

: The last axels position on the bridge causing the largest rotations of the abutments, Type 2 vehicles [m]

\[ a_3 = 10.428 \]

: The last axels position on the bridge causing the largest rotations of the abutments, Type 2 vehicles [m]

\[ a_4 = 8.725 \]

: The last axels position on the bridge causing the largest rotations of the abutments, Type 2 vehicles [m]

\[ s = 10 \]

: Number of traffic queues per day at the same time as a truck is crossing the bridge

\[ Tr = 1560 \]

: Number of trucks per day crossing the bridge

\[ \text{Apile} = \left( \frac{\pi}{4} \right) d \left( d - d_{\text{pile}} - 2 \times t_{\text{pile}} \times 0.001 \right)^2 \]

: [m^2] Area of a pile

\[ \text{POS}(p-(s/Tr)) \]

: [1] POS(p-(s/Tr))
Simulation of strain cycles due to traffic loads – H-piles

**Input distributions**

- **W**: Truck weight probability distribution
- **q**: Queue weight probability distribution
- **p**: Uniform distribution between 0 and 1
- **t**: Calculation step number

**Code**

\[
\text{Str} = \text{POS}(2^\text{b+1}+1)\times \text{Str}1+(1-\text{POS}(2^\text{b+1}+1))\times \text{Str}2
\]

\[
\text{b} = \text{int}(\text{t}/2)
\]

\[
\text{V1} = (\text{Apile}(\text{E}) + (3\times \text{rpm}(\text{Leh}^2))\times \text{Rtot} + (2\times \text{rpm}(\text{Lem})\times \text{Rtot})
\]

\[
\text{Rtot} = \text{POS}(\text{O}+\text{Rsoilmax})\times (\text{O}+\text{Rsoilmax}) + (1-\text{POS}(\text{O}+\text{Rsoilmax}))\times 0
\]

\[
\text{O} = \text{Z} + (\text{qA}\times 1000\times \text{L}^3)/(24\times \text{E}\times \text{I})
\]

\[
\text{V1} = (\text{N}+(\text{qA}\times 1000\times \text{L})/2)/6+(\text{N}+(\text{qA}\times 1000\times \text{L})/2)\times 1000^2, 1/(12,7)
\]

\[
\text{V2} = (\text{N}+(\text{qA}\times 1000\times \text{L})/2)/6-(\text{N}+(\text{qA}\times 1000\times \text{L})/2)\times 1000^2, 1/(12,7)
\]

\[
\text{Z} = \text{POS}(\text{W-m1})\times \text{Za}+(1-\text{POS}(\text{W-m1}))\times \text{R1}
\]

\[
\text{Za} = \text{POS}(\text{W-m2})\times \text{Zb}+(1-\text{POS}(\text{W-m2}))\times \text{R2}
\]

\[
\text{Zb} = \text{POS}(\text{W-m3})\times \text{R4}+(1-\text{POS}(\text{W-m3}))\times \text{R3}
\]

\[
\text{R1} = (\text{W}\times 10000/3)\times (\text{L}-\text{a1})\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000/3)\times (\text{L}-\text{a1}-1,5)\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000/3)\times (\text{L}-\text{a1}-7,5)\times (\text{L}/6\times \text{E}\times 1)^2
\]

\[
\text{R2} = (\text{W}\times 10000\times 15/66)\times (\text{L}-\text{a2})\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000\times 18/66)\times (\text{L}-\text{a2}-7,5)\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000\times 18/66)\times (\text{L}-\text{a2}-9,5)\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000\times 18/66)\times (\text{L}-\text{a2}-11,5)\times (\text{L}/6\times \text{E}\times 1)^2
\]

\[
\text{R3} = (\text{W}\times 10000\times 12/55)\times (\text{L}-\text{a3})\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000\times 12/55)\times (\text{L}-\text{a3}-5)\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000\times 11/55)\times (\text{L}-\text{a3}-8)\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000\times 11/55)\times (\text{L}-\text{a3}-11)\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000\times 11/55)\times (\text{L}-\text{a3}-14)\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000\times 11/55)\times (\text{L}-\text{a3}-17)\times (\text{L}/6\times \text{E}\times 1)^2
\]

\[
\text{R4} = (\text{W}\times 10000\times 11/75)\times (\text{L}-\text{a4})\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000\times 11/75)\times (\text{L}-\text{a4}-1)\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000\times 11/75)\times (\text{L}-\text{a4}-4,5)\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000\times 11/75)\times (\text{L}-\text{a4}-8)\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000\times 11/75)\times (\text{L}-\text{a4}-12,5)\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000\times 11/75)\times (\text{L}-\text{a4}-16)\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000\times 11/75)\times (\text{L}-\text{a4}-20,5)\times (\text{L}/6\times \text{E}\times 1)^2
\]

\[
\text{N1} = (\text{W}\times 10000/3)\times (\text{L})\times (\text{L}+\text{W}\times 10000/3)\times (\text{L}-\text{a1}-1,5)\times (\text{L}) + (\text{W}\times 10000/3)\times (\text{L}-\text{a1}-7,5)\times (\text{L}) + (\text{W}\times 10000/18/66)\times (\text{L}-\text{a2}-7,5)\times (\text{L}) + (\text{W}\times 10000/18/66)\times (\text{L}-\text{a2}-9,5)\times (\text{L}) + (\text{W}\times 10000/18/66)\times (\text{L}-\text{a2}-11,5)\times (\text{L}) + (\text{W}\times 10000/11/55)\times (\text{L}-\text{a3})\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000/11/55)\times (\text{L}-\text{a3}-5)\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000/11/55)\times (\text{L}-\text{a3}-8)\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000/11/55)\times (\text{L}-\text{a3}-11)\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000/11/55)\times (\text{L}-\text{a3}-14)\times (\text{L}/6\times \text{E}\times 1)^2 + (\text{W}\times 10000/11/55)\times (\text{L}-\text{a3}-17)\times (\text{L}/6\times \text{E}\times 1)^2
\]

\[
\text{Ap} = 6641\times 0.001^2 \quad \text{(varying parameter)}
\]
\[ q_A = \text{POS}(p-(s/1560)) \times 0 + (1-\text{POS}(p-(s/1560))) \times q \]

IF \( p \) is larger than \( s/1560 \) THEN the queue weight is zero otherwise \( q \)

\[ b_p = \text{bile} - 2 \times \text{corr} \times 0.001 \]

\([\text{m}]\) Pile width after corrosion

\[ b_{pile} = 0.126 \]

\([\text{m}]\) Pile width

\[ \text{corr} = 0 \]

(varying parameter)

\[ R_{soilmax} = -0.00151 \]

\([-\text{]}\] Max reduction of the rotations due to the backfill soil pressure.

\[ L = 40 \]

Bridge length \([\text{m}]\)

\[ E = 210 \times 10^9 \]

Steel E-modulus \([\text{Pa}]\)

\[ I = 0.0987 \]

\([\text{m}^4]\) Moment of inertia for the superstructure, short term loading

\[ m_3 = 55 \]

Weight limit for Type 3 vehicles \([\text{ton}]\)

\[ m_2 = 38 \]

Weight limit for Type 2 vehicles \([\text{ton}]\)

\[ m_1 = 28 \]

Weight limit for Type 1 vehicles \([\text{ton}]\)

\[ H = 1.65 \]

Abutment height \([\text{m}]\)

\[ \text{Lem} = 4.032 \]

(varying parameter)

\([\text{m}]\) Equivalent cantilever length moment

\[ \text{Leh} = 4.011 \]

(varying parameter)

\([\text{m}]\) Equivalent cantilever length lateral stiffness

\[ a_1 = 14.133 \]

The last axels position on the bridge causing the largest rotations of the abutments, Type 1 vehicles \([\text{m}]\)

\[ a_2 = 12.625 \]

The last axels position on the bridge causing the largest rotations of the abutments, Type 2 vehicles \([\text{m}]\)

\[ a_3 = 10.428 \]

The last axels position on the bridge causing the largest rotations of the abutments, Type 2 vehicles \([\text{m}]\)

\[ a_4 = 8.725 \]

The last axels position on the bridge causing the largest rotations of the abutments, Type 2 vehicles \([\text{m}]\)

\[ s = 10 \]

Number of traffic queues per day at the same time as a truck is crossing the bridge

\[ Tr = 1560 \]

Number of trucks per day crossing the bridge
Simulation of temperature induced strains – pipe piles

**Input distributions**

<table>
<thead>
<tr>
<th>Variable</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nmax</td>
<td>Normal distributed daily max temperature</td>
</tr>
<tr>
<td>Nmin1</td>
<td>Normal distributed daily min temperature during the month with tm&gt;0</td>
</tr>
<tr>
<td>Nmin2</td>
<td>Normal distributed daily min temperature during the month with tm&lt;0</td>
</tr>
<tr>
<td>TGn</td>
<td>Normal distributed negative temperature gradient</td>
</tr>
<tr>
<td>TGp</td>
<td>Normal distributed positive temperature gradient</td>
</tr>
<tr>
<td>t</td>
<td>Calculation step number</td>
</tr>
</tbody>
</table>

**Code**

\[
\text{Str} = \left( \frac{N\text{dead}}{A_{\text{pile}}E}\right) + \left(3\frac{d}{L_{\text{em}}} \right) \left(D_{\text{at}}+H(R_{\text{tg}}+R_{\text{dead}})\right) + \left(2\frac{d}{L_{\text{eh}}} \right) \left(R_{\text{tg}}+R_{\text{dead}}\right) + \text{STRtraff} \\
\text{N\text{dead}} = 1000\left(\frac{P_{\text{ave}}+\text{Conc}+\text{Barr}+\text{Steel}\_2}{L/2} - \left(D_{\text{at}}+H(R_{\text{tg}}+R_{\text{dead}})\right) \right) \\
R_{\text{dead}} = \left(\frac{B_{\text{arr}}+P_{\text{ave}}}{1000}L^3/(24EIST)\right) + \text{Shrink} \\
\text{Shrink} = \left(0.25\frac{1}{10^3}\frac{E_{\text{conc}}}{3}A_{\text{conc}}(e_{LT}-e_{\text{deck}})\right)\frac{L}{2EILT} \\
R_{\text{tg}} = \left(\frac{1}{10^5}\frac{Y}{Ag}\left(e_{LT}-e_{ST}\right)\right)\frac{L}{6IST} \\
\text{STRtraff} = \text{POS}(2b-t+1)\text{STRtraff}99 + (1-\text{POS}(2b-t+1))0 \\
Y = \text{POS}(2b-t+1)\text{TGn} + (1-\text{POS}(2b-t+1))\text{TGp} \\
D_{\text{at}} = \left(\frac{1}{10^5}\frac{L^2}{ETB-To}\right) \\
\text{ETB} = 4.098X \\
X = \text{POS}(2b-t+1)\text{Tmax} + (1-\text{POS}(2b-t+1))\text{Tmin} \\
b = \text{int}(t/2) \\
\text{Tmax} = \text{Tmaxamp}\sin(2\pi t/730-\text{tomax}) + \text{Tmaxmean} + \text{Nmax} \\
\text{Tmin} = \text{Tminamp}\sin(2\pi t/730-\text{tomin}) + \text{Tminmean} + \text{Nmin} \\
\text{Nmin} = \text{POS}(t)\text{Nmin1} + (1-\text{POS}(t))\text{Nmin2} \\
\text{Tm} = \text{Tmamp}\sin(2\pi t/730)\text{Tmmean} \\
\text{Apile} = \left(\frac{\pi}{4}\frac{(d^2-(d_{\text{ypile}}-2\text{tpile}\times0.001)^2)}{}\right) \\
d = \text{dypile}-2\times\text{corr}\times0.001 \\
\text{dypile} = 0.1683 \\
\text{tpile} = 10 \\
\text{corr} = 2.4 \text{ (varying parameter)} \\
\text{L} = 40 \text{ (varying parameter)} \\
\text{To} = 10 \text{ (varying parameter)} \\
\text{Leh} = 2.773 \text{ (varying parameter)} \\
\text{Ag} = 0.1143 \text{ (varying parameter)} \\
\text{eg} = 0.816 \text{ (varying parameter)} \\
\text{e_{deck}} = -0.122 \text{ (varying parameter)} \\
\text{e_{ST}} = 0.210 \text{ (varying parameter)} \\
\text{e_{LT}} = 0.521 \text{ (varying parameter)} \\
\text{IST} = 0.0987 \text{ (varying parameter)} \\
\text{ILT} = 0.0710 \text{ (varying parameter)} \\
\text{H} = 2.25 \text{ (varying parameter)} \\
\text{n} = 6 \text{ (varying parameter)} \\
\text{E} = 210 \text{ (varying parameter)} \\
\text{E}_{\text{conc}} = 35 \text{ (varying parameter)} \\
\text{Aconc} = 1.5 \text{ (varying parameter)} \\
\text{Barr} = 0.5 \text{ (varying parameter)} \\
\text{Pave} = 11.5 \text{ (varying parameter)} \\
\text{Conc} = 37.5 \text{ (varying parameter)} \\
\text{Steel}_1 = 9.7 \text{ (varying parameter)} \\
\text{Steel}_2 = 10.7 \text{ (varying parameter)} \\
\text{Lmid} = 18 \text{ (varying parameter)} \\
\text{Txmaxamp} = 14.9 \text{ (varying between locations)} \\
\text{Tnminamp} = 14.5 \text{ (varying between locations)} \\
\text{Tnmax} = 14.4 \text{ (varying between locations)} \\
\text{tomax} = 0.98 \text{ (varying between locations)} \\
\text{tomin} = 0.90 \text{ (varying between locations)} \\
\text{tom} = 1.02 \text{ (varying between locations)} \\
\text{Tnmaxmean} = 4.2 \text{ (varying between locations)} \\
\text{Tnminmean} = -4.9 \text{ (varying between locations)} \\
\text{Tnmean} = -1.6 \text{ (varying between locations)} \\
\text{STRtraff}99 = 0.00030749 \text{ (varying between load models)} \\
\text{STRtraff}99 = 0.00030749 \text{ (varying between load models)} \\
\text{STRtraff}99 = 0.00030749 \text{ (varying between load models)} \\
\text{STRtraff}99 = 0.00030749 \text{ (varying between load models)}
Simulation of temperature induced strains – H-piles

**Input distributions**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nmax</td>
<td>Normal distributed daily max temperature</td>
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<td>Nmin1</td>
<td>Normal distributed daily min temperature during the month with tm&gt;0</td>
</tr>
<tr>
<td>Nmin2</td>
<td>Normal distributed daily min temperature during the month with tm&lt;0</td>
</tr>
<tr>
<td>TGn</td>
<td>Normal distributed negative temperature gradient</td>
</tr>
<tr>
<td>TGp</td>
<td>Normal distributed positive temperature gradient</td>
</tr>
<tr>
<td>t</td>
<td>Calculation step number</td>
</tr>
</tbody>
</table>

**Code**

```plaintext
Code

Str = (Ndead/(Apile*E*10^9))+(3*bp/((Leh)^2))*(Dabt +H*(Rtg+Rdead)) +
(2*bp/Lem)*(Rtg+Rdead) + STRtraff ; Strain calculations
Ndead = 1000*((Pave+Conc+Barr+Steel2)*L/2 – (Steel2-Steel1))*(L-Lmid)*0,5*(1-(L-Lmid)*0,5/(2*L)) –
(Steel2-Steel1)*((L-Lmid)*0,5/2*(L)^2)/n) ; Normal force in the piles due to deadloads [N]
Rdead = ((Barr+Pave)*1000*L^3/(24*E*IST*10^9))+Shrink  ; Rotations of due to deadloads, shrinkage included
Shrink = 3*(0,25*(1/10^3)*Econc/3)*Aconc*(eLT-edeck)*L/(2*E*ILT)  ; Rotation of the abutments due to shrinkage
Rtg = ((1/10^5)*Y*Ag*(eg-cST)*L^2)/6*IST    ; Rotation due to thermal gradients
STRtraff = POS(2*b-t+1)*STRtraff99+(1-POS(2*b-t+1))*0    ; Traffic load added at daily max temperature
Y = POS(2*b-t+1)*TGn+(1-POS(2*b-t+1))*TGp     ; Every second value are max and min
Dabt = (1/10^5)*L*(EBT-To)/2     ; Lateral displacements of the abutments [m]
EBT = 4+0,98*X      ; Effective Bridge Temperature [°C]
X = POS(2*b-t+1)*Tmax+(1-POS(2*b-t+1))*Tmin  ; Every second value are max and min
tmax = Tmaxamp*sin(2*Pi*t/730-tomax)+Tmaxmean+Nmax    ; daily maximum temperature [°C]
tmin = Tminamp*sin(2*Pi*t/730-tomin)+Tminmean+Nmin   ; daily minimum temperature [°C]
Nmin = POS(Tm)*Nmin1+(1-POS(Tm))*Nmin2    ; Every second value are taken from Nmin1 and Nmin2
Tm = Tmamp*sin(2*Pi*t/730)+Tmmean    ; daily mean temperature [°C]
Apile = 6641*0,001^2   ; Area of a pile [m2]
bpt = bpile corr*0,001  ; Pile width after corrosion [m]
bPILE = 0,1683     ; Pile width [m]
corr = 2,4  ; Depth of the corrosion [mm]
L = 40    ; Length of the bridge [m]
To = 10    ; Temperature at the construction day [°C]
Lem = 2667  ; Equivalent cantilever length - moment[m]
Leh= 2,376 (varying parameter) ; Equivalent cantilever length – lateral stiffness [m]
Ag = 0,1143  ; Cross-section area concrete deck [m2]
eg = 0,816  ; Distance between the top of the upper flange and the center of gravity for the steel girders [m]
edeck = -0,122  ; Distance between the top of the upper flange and the center of gravity for the concrete deck [m]
cST = 0,210  ; Cross-section area concrete girders [m2]
elT = 0,521  ; E-modulus concrete [GPa]
IST = 0,0987 ; Moment of inertia - short term loading [m4]
ILT = 0,0710 ; Moment of inertia - long term loading [m4]
H = 2,25    ; Height of the abutment [m]
n = 6     ; Number of piles at each abutment
E = 210    ; Concrete [kN/m]
Econc = 35 ; Steel end girders [kN/m]
Barr = 0,5 ; Steel midspan girders [kN/m]
Pave = 11,5  ; Steel end girders [kN/m]
Conc = 37,5 ; Steel midspan girders [kN/m]
Steel1 = 9,7
Steel2 = 10,7
Lmid = 18   ; Length of the midspan girders [m]
Tmaxamp = 14,9 (varying parameter) ; Tmax amplitude
Tminamp = 14,5 (varying between locations) ; Tmin amplitude
tomax = 0,98 (varying between locations) ; to max
tomin = 0,90 (varying between locations) ; to min
Tmaxmean = 4,2 (varying between locations) ; Tmax mean value
Tminmean = -4,9 (varying between locations) ; Tmin mean value
STRtraff99 = 0,00030749 (varying between load models) ; Added strain from traffic load model
```
Appendix M

Relationship Between Allowable Gross Weight and Axel Distances

The information in this appendix can be found in the Swedish Traffic Regulation, Trafikförordningen (1998:1276).

Table M:1 Allowable gross weight for vehicles with different axel distances.

<table>
<thead>
<tr>
<th>Distance between the first and the last axel</th>
<th>More than [m]</th>
<th>Less than [m]</th>
<th>Allowable gross weight [ton]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1</td>
<td>11.5</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1.3</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>1.3</td>
<td>1.8</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>1.8</td>
<td>2</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>2.6</td>
<td>21</td>
<td></td>
</tr>
<tr>
<td>2.6</td>
<td>5</td>
<td>24</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>5.2</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>5.2</td>
<td>5.4</td>
<td>26</td>
<td></td>
</tr>
<tr>
<td>5.4</td>
<td>5.6</td>
<td>27</td>
<td></td>
</tr>
<tr>
<td>5.6</td>
<td>5.8</td>
<td>28</td>
<td></td>
</tr>
<tr>
<td>5.8</td>
<td>6</td>
<td>29</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>6.2</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>6.2</td>
<td>6.4</td>
<td>31</td>
<td></td>
</tr>
<tr>
<td>6.4</td>
<td>8.25</td>
<td>32</td>
<td></td>
</tr>
<tr>
<td>8.25</td>
<td>8.5</td>
<td>33</td>
<td></td>
</tr>
<tr>
<td>8.5</td>
<td>8.75</td>
<td>34</td>
<td></td>
</tr>
<tr>
<td>8.75</td>
<td>9</td>
<td>35</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>9.25</td>
<td>36</td>
<td></td>
</tr>
<tr>
<td>9.25</td>
<td>9.5</td>
<td>37</td>
<td></td>
</tr>
<tr>
<td>9.5</td>
<td>9.75</td>
<td>38</td>
<td></td>
</tr>
<tr>
<td>9.75</td>
<td>10</td>
<td>39</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>10.25</td>
<td>40</td>
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<td>10.25</td>
<td>10.5</td>
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</tr>
<tr>
<td>10.5</td>
<td>10.75</td>
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<td></td>
</tr>
<tr>
<td>10.75</td>
<td>11</td>
<td>43</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>11.25</td>
<td>44</td>
<td></td>
</tr>
<tr>
<td>11.25</td>
<td>11.5</td>
<td>45</td>
<td></td>
</tr>
<tr>
<td>11.5</td>
<td>11.75</td>
<td>46</td>
<td></td>
</tr>
<tr>
<td>11.75</td>
<td>12</td>
<td>47</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>12.5</td>
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<td>15</td>
<td>53</td>
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</tr>
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<td>15</td>
<td>15.5</td>
<td>54</td>
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<td>15.5</td>
<td>16</td>
<td>55</td>
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</tr>
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<td>17</td>
<td>57</td>
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</tr>
<tr>
<td>17</td>
<td>17.5</td>
<td>58</td>
<td></td>
</tr>
<tr>
<td>17.5</td>
<td>18</td>
<td>59</td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>24</td>
<td>60</td>
<td></td>
</tr>
</tbody>
</table>

Figure M:1 Relationship between the allowable gross weight and the total axel distance.
Low-cycle Fatigue of Steel Piles in Integral Abutment Bridges
Appendix N

Maximum Integral Bridge Lengths in 30 States in the USA

Table N:1 Length limits and skew angle limitations in 30 states in USA (Bakeer et al. 2005)

<table>
<thead>
<tr>
<th>State</th>
<th>First year built</th>
<th>Length limit [m]</th>
<th>Skew angle [degrees]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arkansas</td>
<td>1996</td>
<td>79</td>
<td>33</td>
</tr>
<tr>
<td>California</td>
<td>1959</td>
<td>25 mm movement</td>
<td>45</td>
</tr>
<tr>
<td>Georgia</td>
<td>1975</td>
<td>125 / 79</td>
<td>0 / 40</td>
</tr>
<tr>
<td>Hawaii</td>
<td>-</td>
<td>76</td>
<td>-</td>
</tr>
<tr>
<td>Illinois</td>
<td>1983</td>
<td>92</td>
<td>30</td>
</tr>
<tr>
<td>Indiana</td>
<td>-</td>
<td>92</td>
<td>30</td>
</tr>
<tr>
<td>Idaho</td>
<td>-</td>
<td>122</td>
<td>30</td>
</tr>
<tr>
<td>Iowa</td>
<td>1962</td>
<td>92</td>
<td>30</td>
</tr>
<tr>
<td>Kansas</td>
<td>1935</td>
<td>137</td>
<td>-</td>
</tr>
<tr>
<td>Kentucky</td>
<td>1970</td>
<td>122</td>
<td>30</td>
</tr>
<tr>
<td>Louisiana</td>
<td>1989</td>
<td>305</td>
<td>0</td>
</tr>
<tr>
<td>Maine</td>
<td>1983</td>
<td>46</td>
<td>30</td>
</tr>
<tr>
<td>Michigan</td>
<td>1990</td>
<td>-</td>
<td>30</td>
</tr>
<tr>
<td>Missouri</td>
<td>-</td>
<td>183</td>
<td>-</td>
</tr>
<tr>
<td>Massachusetts</td>
<td>1930</td>
<td>92</td>
<td>30</td>
</tr>
<tr>
<td>North Dakota</td>
<td>1960</td>
<td>122</td>
<td>30</td>
</tr>
<tr>
<td>Nevada</td>
<td>1980</td>
<td>61</td>
<td>45</td>
</tr>
<tr>
<td>New York</td>
<td>1980</td>
<td>92</td>
<td>30</td>
</tr>
<tr>
<td>Ohio</td>
<td>-</td>
<td>114</td>
<td>30</td>
</tr>
<tr>
<td>Oklahoma</td>
<td>1980</td>
<td>64</td>
<td>0</td>
</tr>
<tr>
<td>Pennsylvania</td>
<td>1946</td>
<td>183</td>
<td>20</td>
</tr>
<tr>
<td>Oregon</td>
<td>1940</td>
<td>61</td>
<td>25</td>
</tr>
<tr>
<td>South Dakota</td>
<td>1948</td>
<td>214</td>
<td>35</td>
</tr>
<tr>
<td>South Carolina</td>
<td>-</td>
<td>153</td>
<td>30</td>
</tr>
<tr>
<td>Tennessee</td>
<td>1965</td>
<td>50 mm movement</td>
<td>No limit</td>
</tr>
<tr>
<td>Utah</td>
<td>-</td>
<td>92</td>
<td>20</td>
</tr>
<tr>
<td>Virginia</td>
<td>1982</td>
<td>153</td>
<td>-</td>
</tr>
<tr>
<td>Wyoming</td>
<td>1957</td>
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<td>1965</td>
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<tr>
<td>Wisconsin</td>
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<td>92</td>
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