Secondary Strain in Web Stiffeners in Steel and Composite Bridges

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Licentiate Thesis

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Abstract

In the summer of 2006 numerous cracks were found at a highway bridge near Stockholm in Sweden. All cracks occurred at welds connecting vertical web stiffeners to the top flanges of the main girders. In order to identify the cause behind the cracks, an investigation was conducted including the following:

- Bridge monitoring
- Rain-flow Counting- and cumulative damage index evaluations
- Finite element analyses
- Analyses of a piece of a weld containing a crack, using optical- and scanning electron microscope
- A number of static tests and one cyclic test were performed upon a composite bridge at Luleå University of Technology (LTU).

The result of the investigation is unambiguously indicating fatigue as being the cause of the cracks. The fatigue relevant stress is believed to be the result of imposed (or forced) rotations of stiff cross girders and web stiffeners. The rotations occur when the slender concrete deck deflects for heavy vehicles passing by. This to its nature secondary effect was probably overlooked at the design phase of the bridge.

In addition, tests were performed on a small composite bridge with two kinds of cross girders, at Luleå University of Technology (LTU). The tests and their results are described in the thesis, as well as the assessments of Vårby Bridge.

Based on the results obtained during the investigation of the cracks at the highway bridge and the lab tests at LTU, a few detailing recommendations are made for bridge designers. A proposal of a method for evaluation of fatigue critical stress in stiffeners is also presented.

Keywords: Fatigue, Web stiffener, Cross girder, Composite bridge, Deformation induced fatigue, Fillet weld, Butt weld, Palmgren-Miner
Sammanfattning


- Töjningsmätningar på bron.
- Delskadeberäkningar enligt Palmgren-Miner och utförda på basis av långtidsmätningarna
- Finita element analyser
- Mikroskopanalyser av bit av en svets med en sprickyta
- Tester utförda på en mindre samverkansbro vid Luleå Tekniska Universitet (LTU)


Tester på en mindre samverkansbro har utförts vid Luleå tekniska Universitet. Två olika typer av tvärförband testades. Testerna och resultatet av dessa beskrivs i uppsatsen.

På basis av resultatet av Vårby-utredningen samt av utförda tester vid LTU, och med ett syfte att minska sannolikheten för liknande sprickbildningar, så har ett antal detaljutförningar utvärderats. Vidare så föreslås en enklare metod för utvärderingar av kritiska spänningsvidder i livavstyvningar.

Nyckelord: Utmattning, Livavstyving, Tvär balk, Tvär förband, Samverkansbro, Deformationsst理赔 utmattning, Kälsvets, Stensvets, Palmgren-Miner
Preface

The work presented in this thesis was carried out at the Division of Structural and Construction Engineering, department of Civil, Mining and Natural Resources Engineering at Luleå University of Technology (LTU) and at Ramböll.

The investigation related to cracked welds at the Vårby Bridge and described in the thesis was carried out under the commission of Kenth Jansson at Trafikverket (the Swedish road & railway administration). The same investigation was also implemented in a European research project called BRIFAG (Bridge Fatigue Guidance). The project was carried out with a financial grant from the research fund for Coal and Steel of the European Community, granted under the contract # RFSR-CT-2008-00033.

Funding has also been granted from the following organisations and to whom I am grateful:

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- Banverket: the former Swedish Railway Administration, now implemented in Trafikverket

At a professional level, my deepest gratitude and appreciation goes to the following persons:

- My supervisor, inspirer, colleague and friend Professor Peter Collin. Thank you for all favours you have done for me during the years
- The rest of my colleagues at the bridge design section at the Ramböll office in Luleå. It is a privilege and my honour to work with and learn from you all
- Docent Kjell Eriksson. A man with knowledge in the topics of fatigue and fracture mechanics that not many can match
Gerard James, the man in charge of the monitoring of the Vårby Bridge

All personnel at COMPLAB who performed the testings’ upon a composite bridge at LTU in the summer of 2011

At Chalmers University of Technology and as a part of a master’s thesis, finite element analyses related to Vårby Bridge were conducted by Robert Bengtsson and Mikael Widén and under the guidance of Docent Mohammad Al-Emrani

At a more personal level, a warm thought of appreciation goes to my dear mother and father. The best parents a son could have ever wished for. Thank you.

Last but not least, to my family; my fiancée Johanna, my daughter Elise and bonus-son Kristoffer; merely the joy of being in your presence every day makes me the happiest man in the world. Love, always.

Luleå, September 2012

Mattias Nilsson
Synopsis

In the summer of 2006, numerous cracks were found in Vårby Bridge; a highway bridge crossing the bay of Vårby near Stockholm in Sweden. All the cracks were in welds connecting single vertical web stiffeners to the top flanges of the main girders. As the affected welds are fillet, rather than butt, welds the cracking appeared only in the weld material and not in the stiffener steel plate material.

In order to identify the causes of the cracks; an investigation commissioned by Trafikverket (the Swedish Road and Railway Administration) was started in the spring of 2009, including, amongst other evaluations, the following:

- Monitoring of the Bridge. Short term monitoring conducted for a vehicle with known weight and position, as well as long-term monitoring to assess effects of daily traffic flows
- Rain-flow Counting- and cumulative damage index evaluations
- Finite element analyses
- Analysis of a fracture surface of one of the cracks, using both optical- and scanning electron microscope

The results of the investigation unambiguously indicate material fatigue as being the cause of the cracks. All other possible causes with varying degrees of plausibility can be excluded regarding the following:

- Striations and beach-marks were found at the fracture surface of a crack in a small sample of a weld collected from the bridge. Striations and beach-marks are strong indications of crack growth through fatigue.
- The frequency and magnitude of fatigue-relevant stresses in areas of interest were found to be sufficient to explain the cracking. Indeed, such stress peaks are numerous since the traffic flows are heavy, and the passage of every sufficiently heavy vehicle could contribute to fatigue damage
- The evaluation of the fatigue life of two joints, performed on basis of the long-term monitoring and using more realistic fatigue strength was found to be 7
respectively 14 years. The result seems reasonable considering the fact that the cracks were discovered 10 years after the bridge was taken into service.

The fatigue relevant stress at the origins of the cracks is believed to be the result of imposed (or forced) rotations of cross girders and web stiffeners with high bending stiffness. Such rotations occur when a slender concrete deck deflects as heavy vehicles pass, and were probably due to their secondary nature overlooked during the design of the bridge.

In short, several adverse factors influence Vårby Bridge. The most substantial are listed below:

- **Large rotations of the bridge deck**
  
  As the bridge deck is wide – 14,0 m - and carried by two main girders only, the distance between the girders is consequently also wide. Moreover, to be able to resist governing shear forces in the concrete deck, the thickness of the deck is > 340 mm at a section above the main girders. But as the deck is tapered, the corresponding thickness at a section in the centre between the two main girders is only 280 mm. The thickness of the deck must consequently be considered as being extreme in the sense that such thin deck plates usually are found only at smaller composite bridges where the distance between the main girders are in the range of perhaps 3 m.

  Due to the combination of the large distance between the main girders and the relatively thin deck plate, the ratio between the span length of the deck and the deck thickness is large. A large ratio (or slenderness) implies large deflections when heavy vehicles pass, and consequently large rotations.

- **Large restraint**
  
  Both vertical stiffeners and intermediate cross girders have greater bending stiffness than is appropriate for such members. The main reason for that is that the cross girders were designed to support the scaffolding when the bridge deck was cast. Furthermore, and for the same reason, the distance between the top surface of the main girders and the corresponding top surfaces of all cross girders is short, just 400 mm. The cross-sections of interest are consequently short and stiff, i.e. a particularly poor combination for structural members exposed to large, cyclic rotations.

- **Inadequate fatigue strength**
  
  The secondary effects discussed in this thesis were not anticipated during the design of the bridge. Consequently, the cracked welds are all fillet welds rather than butt welds, which have greater fatigue strength. Furthermore, the welds
were only designed to withstand shear force, thus the fillet welds are only half as wide as the stiffeners

- **Large daily traffic flow**

  By Swedish standards, Vårby Bridge carries extremely heavy traffic. Each of the two adjacent bridges carries more than 40,000 vehicles daily, including numerous heavy vehicles, all of which may contribute to fatigue damage.

  For example, during the long-term measurement campaign (104 hours in total) approximately 11,700 peaks > 10 MPa were registered. Extrapolated over a period of 120 years — the expected lifetime of the bridge — the corresponding number of peaks > 10 MPa exceeds 100 million. In future, for similar joints of bridges carrying extremely heavy traffic, use of a design criteria equal to the intersection of the fatigue strength curve \((m = 5)\) at \(10^6\) cycles could be considered.

Most stiffener stresses at points of interest were found to be compressive. As fatigue cracks usually form and grow in areas where the resultant stress is tensile, the presence of residual stresses at the origin of the cracks is assumed. Further, the direction of the residual stress field is assumed to be perpendicular to the crack planes. Research results supporting both assumptions are presented in the thesis. It is believed that the mean stress in fatigue-relevant stress cycles imposes high tension due to high residual stress at weld toes adding to the normally nominal compressive stress caused by live loads.

The cracks are believed to constitute no immediate hazard for the safety and sustainability of the bridge. However, in a worst-case scenario, if the propagation continues the stiffeners could in time become separated from the top flanges. If so, out-of-plane bending of the main girder web plates could occur, leading to "moustache cracks" in the main girder web plates. This would be highly undesirable and must be prevented. But as stated earlier, the crack growth in Vårby Bridge is believed to be due to the deflection of the concrete deck. This implies that the cracking is probably governed more by displacement than by load. In contrast to a case where the cracking is governed by load; the stress intensity, a critical factor for the crack growth rate, should decrease as the length of the cracks increases, thereby retarding further crack growth. Hence it is believed that propagation of the observed cracks will slow and eventually stop. However, the assumption that cracking will slow, has not been tested further, by mechanical fracture analysis for example. Therefore, during the installation of the monitoring equipment, a crack was selected and marked to allow its further growth to be followed, and it is recommended that the bridge owner continues to inspect it, tentatively at least once a year.

Attempts to reproduce the structural behaviour of the bridge through finite element analyses satisfactorily modelled the global behaviour of the superstructure. Regarding
stiffener stress at the origins of the cracks, it was concluded that the finite element model represents the behaviour of the real structure accurately in principle, but the response of the model as a whole is far too rigid and yields far higher stresses than corresponding measured stresses.

In the thesis, attention is paid to the impact of the concrete shrinkage on the vertical stiffeners. This is not due to a suspicion that the shrinkage is the cause of the cracks. On the contrary, the shrinkage was swiftly excluded as a probable cause. However, some ambiguities found in the short-term monitoring data that are not otherwise easy to explain could possibly be due to effects related to shrinkage of the concrete deck. It was found that the following possibilities cannot be excluded:

- The shrinkage could possibly cause local gaps between the deck and the steel top flanges at locations of vertical stiffeners. Such gaps or interstices have been found at cross girder-connections in the Vårby Bridge and other similar bridges. It is suggested that since heavy vehicles pass between the two main girders such gaps could reduce negative nominal stresses at the top corners of stiffeners due to the rotation imposed on them. When attempts were made to reproduce gaps in the FE-analyses, better concurrence with the measurements was found.
- Given a shear connector very near a vertical stiffener; it is possible that tensile forces could be introduced into a vertical stiffener that is very near or aligned with a shear connector, via the shear connector and associated weld. Such forces would cause a permanent tensile stress field perpendicular to the weld and consequently to the crack planes. It is assumed that such a steady state stress would affect the fatigue resistance of the particular weld negatively as, together with residual stresses, it would raise the mean stress of a typical fatigue stress cycle.

Lab tests were performed upon a small composite bridge at Luleå University of Technology (LTU) in the summer of 2011. A large number of strain gauges were installed and four static tests were performed on two types of cross girders, both of which are commonly used in composite bridges in Sweden. In the same manner as for the Vårby Bridge, the measured data were compared to corresponding data obtained from FE analyses. A better concurrence between the measurements and FE analyses was found.

Several strain gauges were placed on chosen shear connectors in the vicinity of one of the vertical stiffeners. It was found that, in particular a shear connector in alignment with the vertical stiffener was subjected to both a substantial axial force and multi-axial bending. As shear connectors are not intended to transfer large repetitive axial forces, it is suggested that attention should be paid to both the longitudinal and
transversal positions of the shear connectors in relationship to vertical stiffeners when designing future composite bridges.

The static tests were complemented with a cyclic test on one of the cross girders, in which approximately 1.14 million cycles were applied (the maximum possible under time constraints). The main focus during the cyclic test was on data collected from a strain gauge situated at one of the vertical stiffeners. When evaluating the cumulative damage in the weld connecting the stiffener to the main girder top flange, the results indicated a very high probability for the appearance of a crack. However, no crack appeared.

Based on the analysis of the cracks in Vårby Bridge and the lab tests performed at LTU, it is suggested that designers of future steel and/or composite bridges should consider the following:

- An enhancement of the fatigue strength: Welds between stiffeners and main girder top flanges should preferably be butt welds rather than fillet welds. This simple measure - provided a fillet weld area equal to the cross-sectional area of the stiffener - should raise the fatigue strength by a factor of two and prolong the fatigue life of the weld by a factor of eight.

- An elimination of stress concentration: The stress concentration at the end of stiffeners should preferably be reduced by including a soft transition between the stiffeners and flanges.

- A reduction of restraint: Bracing elements near joints of interest should, if possible, be excluded. If that is not an option, the connections between the bracing elements and stiffeners should be as flexible as possible.

It is also suggested that attention should be paid to both longitudinal and transversal positions of shear connectors relative vertical web stiffeners. In order to avoid introducing tensile forces into main girders at locations of vertical stiffeners, shear connectors could be positioned in pairs with a narrow transversal spacing. Moreover, shear connectors near, or even directly aligned with vertical web stiffeners are not recommended.

It is left to the bridge designers to estimate the necessity of each of the above suggestions for every individual bridge.

A method for evaluating bending moments in vertical stiffeners subject to imposed (or forced) rotations is suggested. The method should ideally;
• Be simple to use and preferably not involve advanced, time-consuming finite element modelling
• Give results of the same order as corresponding results obtained from advanced Finite Shell Element analyses
• Give conservative results, with a wide enough safety margin.
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**Notations and symbols**

*Roman upper-case letters*

- $A$ cross-sectional area
- $B$ width of a member
- $C$ constant of the Paris law
- $D$ damage index. $0 \leq D \leq 1.0$
- $E$ modulus of elasticity of a material
- $I$ moment of inertia
- $K$ elastic stress intensity factor
- $K_{IC}$ fracture toughness of a material
- $L$ length of a member
- $M$ general denotation for a bending moment
- $M_{\text{max}}$ maximum bending moment
- $M_{\text{min}}$ minimum bending moment
- $N$ number of cycles
- $N_f$ the number of cycles needed from crack initiation to complete failure
- $N_i$ theoretical fatigue life at stress range $i$
- $N_{\text{shrink}}$ axial force in a member due to shrinkage of concrete
- $P$ general denotation for a load
- $S_{\text{flex}}$ elastic flexibility corresponding to the combination of the axial stiffness of shear connectors and to the deformability of flanges for loading perpendicular the plane of the flange
$S_{\text{Cross girder}}$ elastic flexibility corresponding to displacement of a cross girder frame for a unit load

$U$ potential energy or utility ratio

$W$ bending resistance

$Y$ dimensionless parameter in Paris law

**Roman lower-case letters**

$a$ crack length in general

$a_{\text{c}}$ critical crack length

$a_{0}$ initial crack length

$b_{f}$ width of a flange

$g$ either a web gap or a gap between the concrete deck and steel top flange

$m$ exponent of Paris law or the slope of S-N curves

$n_{i}$ actual number of cycles at stress range $i$

$t_{w}$ web thickness

$w$ width of a weld

**Greek letters**

$\alpha$ scalar for converting a stiffener stress to a fillet weld stress. Also the thermal expansion coefficient of steel

$\gamma_{\text{Mf}}$ partial safety factor for fatigue strength

$\varepsilon_{i}$ strain in general

$\varepsilon_{\text{average}}$ average strain in a member
$\varepsilon_{\text{bending}}$ strain due to a bending moment

$\varepsilon_{\text{measure}}$ measured strain

$\varepsilon_{\text{shrink}}$ strain due to shrinkage

$\delta$ generally a displacement

$\Delta$ generally a displacement

$\Delta_{\text{girders}}$ differential deflection of adjacent girders

$\Delta L$ elongation or contraction of a member due to an axial force

$\Delta K$ elastic stress intensity factor range

$\Delta M$ bending moment range

$\Delta \sigma$ nominal stress range

$\Delta \sigma_{\text{axial}}$ stress in a member due to an axial force

$\Delta \sigma_{\text{C}}$ fatigue strength at $2 \cdot 10^6$ cycles

$\Delta \sigma_{\text{E,2}}$ equivalent constant amplitude stress range at $2 \cdot 10^6$ cycles

$\Delta \sigma_{\text{th}}$ threshold value of a stress range

$\Delta \sigma_{\text{weld}}$ stress range in a weld,

$\Delta \sigma_{\text{stiffener}}$ stress range in a stiffener

$\phi$ dynamic amplification factor

$\theta$ general denotation for a rotation

$\theta_{\text{A}}$ rotation due to a differential deflection of adjacent members

$\lambda$ fatigue damage equivalent factor

$\sigma$ general denotation for a stress

$\sigma_{\text{gap}}$ distortional web-gap stress
$\sigma_{FE}$ stress taken from a finite element model

$\sigma_{L, shrink}$ normal stress perpendicular a weld and deduced to shrinkage

$\sigma_{Meas}$ measured stress

$\sigma_{Stiff, shrink}$ stiffener stress due to shrinkage

$\tau_{// shrink}$ shear stress in a weld, parallel to the longitudinal axis of the weld and deduced to shrinkage
1 Introduction

1.1 Aims and scopes

The aim of this thesis was limited to provide answers to the following research questions;

1) What is the true cause of the cracks in Vårby Bridge?

2) Why did the cracks appear at this particular bridge?

3) How will the cracks develop, and is there any immediate hazard concerning the safety and durability of the bridge?

From a more general point of view and for the benefit of future bridges, the following questions are asked;

4) Can stiffener stresses be predicted by Finite Element analysis or manually? How should bridge designers estimate fatigue-relevant stresses?

5) Can bridge designers take measures against the phenomenon that caused the cracks?

The above research questions are discussed in Chapter 7. Furthermore and in the same chapter, a few suggestions are made to future researchers.
1.2 Method
The main source of information for this thesis is the following:

- A performed literature survey
- An international workshop organized by Rambøll, Luleå University of Technology (LTU) and IABSE and held at the Rambøll head office in Stockholm the 4th of March 2010. The topic of the seminar was Strengthening of Steel Bridges. The seminar had thirty-five participants from seven countries speaking and listening. One speaker was Andreas Lechner, Graz University of Technology. A member of the inquiry team in an Austrian investigation on the occasion of cracks at a few Austrian bridges and resembling those found at the Vårby Bridge

- The vast amount of data achieved during the performed bridge monitoring on the occasion of the cracks occurring at the Vårby Bridge
- The testings' performed at LTU, upon a small composite bridge in the summer of 2011

1.3 Limitations
- All finite element analyses described in the thesis are performed in FE-models of a global nature. The FE-models are accordingly not suitable for evaluations of so called “hot-spot” stresses. For such evaluations, a refinement is needed. For instance by a creation of sub-models with sufficiently dense mesh. Existing stress raisers are therefore assumed to be included in the appropriate fatigue detail category
- The remaining fatigue life of the cracked welds at the Vårby Bridge has not been tested by for example fracture mechanic methods
- No refurbishment and/or repairing methods of the cracked weld connections at the Vårby Bridge are suggested
1.4 Disposition of the thesis
The thesis is divided into seven chapters. In the following, a short comprehension of each chapter is made.

Chapter 2 contains a general overview of different kinds of cross girders and diaphragms in steel- and composite bridges. The purposes of such members are discussed. The difference between intermediate cross girders and corresponding at supports is described as well as the difference between the same members in I-girder and box-girder bridges.

Chapter 3 contains a short description of the Vårby Bridge. Both the geometry of the bridge and the traffic situation at the bridge is described.

Later in the chapter, the cracks at the Vårby Bridge are described. The distribution, positions at the stiffeners, positions at the welds, the lengths of- and the propagation rate of the cracks are discussed.

Possible causes of the cracks, more or less likely, are being identified and in some cases also excluded as a probable cause.

Attention has been given to the impact of shrinkage of the concrete deck upon vertical stiffeners and cross girders. This is not due to a suspicion of the shrinkage being the cause of the cracks. On the contrary, the shrinkage was quickly excluded as a probable cause. However, some ambiguities found in the results of the performed measurement could possibly be explained by effects due to the shrinkage.

The chapter ends with an overview of cases with cracks resembling those found at the Vårby Bridge.

Chapter 4 describes the monitoring of the Vårby Bridge, both the short-term monitoring for a vehicle with known parameters as weight and position-, as well as the long term monitoring for daily traffic.

Two of a total of sixteen passages during the short-term monitoring were chosen to be studied more in detail. Obtained stiffener stresses emerging during those two passages are presented and compared to corresponding stresses taken from finite element analyses.

In the same chapter, the results of the long term monitoring are being studied. Based upon Rain-flow Counting analyses, the fatigue damage indexes at points of interest are evaluated.
Introduction

The chapter ends with the presentation of the result of microscope analyses of a small sample of a weld containing a crack.

Chapter 5 is a resume of performed testing’s upon a small composite bridge at LTU, Luleå. The testing’s took place in the summer of 2011. A number of static tests were performed as well as one cyclic test. Results of each test are presented and compared to numerical results.

In Chapter 6, detailing aspects are being discussed. A few common types of cross girder are discussed as well as possible measures to counteract fatigue damages at points of interest.

A simplified method for evaluation of fatigue relevant stress ranges in stiffeners is proposed and illustrated by some examples.

In Chapter 7, attempts to answer the research questions in section 1.1 are made. Furthermore, some suggestions to bridge designers and to future researchers are made.

The thesis also contains two appendixes as described below;

Appendix A:
Rain-flow Counting histograms for strain gauge number 1 to 21, used during the monitoring of the Vårby Bridge

Appendix B:
Exhibition of various types of cross girders from all over the world
2 Web Stiffeners and Cross Girders in Composite Bridges

As the main topics of this thesis are web stiffeners and cross girders in steel- and composite bridges; it is in place to describe the purposes of such members.

As seen in Appendix B, smaller parts within the main frame concept of cross girders are designed in many different ways. But the main concept itself, the design philosophy, is recognized in most examples, no matter the origin or country. Cross girders as the ones exemplified by the standardized drawings from the State of New York could equally as well have been designed in Austria, Germany, Sweden or another equivalent country.

The observation is by itself a guarantee for a reliable and well-proven concept. However, there are small parts within the main-frame concept that, for better sustainability, needs to be highlighted. In this thesis, an attempt is made to do so.

2.1 Cross girders in I-girder Composite bridges

2.1.1 Intermediate cross girders
The main purposes of intermediate cross girders in steel- and composite bridges are;

- During the casting of the deck prevent lateral torsional buckling of top flanges
- Prevent lateral torsional buckling of bottom flanges near mid supports
- Provide resistance against lateral forces as for example wind
- In cases of horizontally curved bridges but piecewise straight main girders, the cross girders must absorb forces that occurs due to the obliquity between connecting assembly units

Where the height of the main girders are limited and the distance between the main girders are not too wide, a typical intermediate cross girder often consist of a single I-
or U-shaped girder, sometimes supported by diagonals, see figure 1, the cross girder to the left.

![Figure 1](image1.png)

*Figure 1 To the left; Bridge crossing the highway E4 at Skulnäs. To the right; Bridge over the river of Nissan.*

In cases where the main girders are of an even lower height, intermediate cross girders can be designed as for example single I-shaped girders, bolted or welded to vertical web stiffeners. It is in such cases advantageous to integrate the cross girders with the concrete deck on top, see figure 1, the cross girder to the right.

Where the height of the main girders is larger, a commonly used type of intermediate cross girders is the so called “K-truss”, usually consisting of a top- and a bottom chord tied together by two diagonals as in the figure below;

![Figure 2](image2.png)

*Figure 2 Cross girder designed as truss. Bridge crossing the lake of Forssjön*

If the distance between the main girders is not too wide, for example as in the case of a railway bridge, the number of diagonals can be limited to one.
Two international examples of K-trusses (Austria & USA) are seen in the two figures below.

**Figure 3 Intermediate cross girder - Austria**  
**Figure 4 Intermediate cross girder. USA**

The cross girders are normally manufactured at work shops as assembly units and are bolted or sometimes welded at the construction site to vertical web stiffeners attached to the main girders. The main purpose of the stiffeners is however to increase the shear force capacity of normally slender main girder web plates.

As concrete decks by common practise have been considered as simply supported upon the main girders, welds between web stiffeners and main girder flanges have consequently been designed to withstand actions of horizontal forces only. Examples are wind, brake and centrifugal force. As the magnitudes of such forces usually are moderate the welds have often been performed as fillet welds and not butt welds.

**Figure 5 Horizontal forces upon cross girders**

Examples of bridge girders without web-stiffeners have also been found. For instance in Texas, USA, where hot-rolled girders with thick, un-stiffened web plates were used in the early 1960’s.

**Figure 6 - 9** show examples of cross girders issued by the Texas Highway department. The standardized drawings were valid for multi-girder highway bridges built in the early 60’s.
The web plates of the main girders were apparently sufficiently thick enough for the allowance of the lack of web stiffeners. When studying the detailing of the above cross girders, it is clear that the concept “fatigue” was probably unknown- or overlooked at the design phase. The welded connections of the diagonals to the top flanges- and to the web plates of the main girders are extremely poor from a fatigue point of view.
2.1.2 Cross girders at supports
The main purposes of cross girders at supports in I-girder composite bridges are to transfer lateral loads like wind, brake loading and centrifugal force to the bearings as well as preventing the bottom flanges of the main girder from lateral-torsional buckling.

The governing load case in the Ultimate Limit State (ULS) for the majority of all cross girders at any support is usually a future exchange of the bearings as hydraulic jacks will be placed against the girders, lifting the whole superstructure.

As an example, the governing lifting forces for the cross girder in the figure above was evaluated as 5,6 MN including appropriate partial load safety factors.

While unburden the bearings beneath the main girders, the cross girder will be subjected to a bending moment equal to 7,3 MNm and a shear force equal to 5,6 MN.

It is evident that no service limit load comes even close to cause sectional forces in the cross girder of such large magnitudes.
2.2 Cross girders in box-girder composite bridges

The main purposes of intermediate cross girders in box-girder composite bridges are basically the same as for corresponding structural elements in I-girder bridges.

One difference between cross girders at I-girder composite bridges and corresponding in box-girder bridges is that the latter must transfer externally applied torque caused by eccentric loading to the box walls through shear flow. Moreover, as the cross section of box girders are flexible in the sense that the shape of the box girder can distort in the cross-wise direction as indicated in figure 11 below, an important objective for cross girders or diaphragms in box girders bridges is to preserve the shape of the box against torsional out-of-plane distortion (warping).

![Figure 11 Out-of-plane bending distortions (warping) of a thin-walled box section. Picture taken from [16]](image)

At supports, and as the bearings often are disposed in between the web plates of the box-girder, one important purpose is to transfer shear force from the web plates to the bearings.
Furthermore, as at least one of the bearings normally is guided or fixed, the cross girders have to transmit horizontal forces into shear flow along the web of the box section.
3 The Occurrence of Cracks at the Vårby Bridge

In the summer of 2006, approximately 10 years after the bridge was taken into service and during a routine inspection, numerous cracks were found at a highway bridge in the vicinity of Stockholm, Sweden. Generally, all cracks occurred at welds connecting vertical web stiffeners to the top flanges of the main girders.

The cracks were all located in the spans of the bridge i.e. at intermediate cross girders attached to single vertical web stiffeners.

No crack has yet, to the knowledge of the author, been observed at supports, where the web plates of the main girders are fitted on both sides with stiffeners.
3.1 Bridge geometry
The Vårby Bridge, a highway bridge in the vicinity of Stockholm consists in fact of two adjacent composite bridges (denoted the North Bridge and the South Bridge).

Each bridge has six spans of length $38 + 4 \times 44 + 38$ m adding up to a total length of 252 m between the end-supports 1 and 7.

![The Vårby Bridge](image1)

The bridges are each composed by two continuous I-shaped steel girders in composite action with the concrete deck.

The height of the two steel girders is varying linearly from a height of 1,5 m in the middle of the spans to a height of 2,0 m at mid supports. The transversal distance between the two steel main girders is 7,5 m.

The concrete deck has a width of 14,0 m ($3,25 + 7,50 + 3,25$ m). As the thickness of the deck centrally between the two main girders is only 0,28 m, the deck must be considered as slender in the sense that the ratio between the width and the thickness of the deck is high.

The composite action between the steel girders and the concrete deck is obtained by headed shear connectors embedded in the concrete.

The shear connectors are distributed in two longitudinal rows on each of the top flanges. The distance between the studs is alternating 150, 190 or 260 mm depending on the governing shear force flow between the steel and the concrete. The transverse spacing between the two rows is kept constant, 550 mm.

The two main girders are further tied together by transverse steel members, so called cross girders, described in the following.
Intermediate cross girders

The two main girders at each bridge are stabilized by I-shaped cross girders in the spans as well as at all supports. The distances between the cross girders are typically between 7.1 and 7.4 m. The upper surfaces of the top flanges are situated 400 mm below the top surface of the main girders as the girders supporting the scaffolding while casting the deck. The cross girders are attached to single vertical web stiffeners by bolts. The stiffeners, 250 x 25 mm² are in turn welded to the web and the flanges of the main girders by fillet welds of width $w = 5$ mm.

Cross girders at mid supports

The cross sections of the girders at mid supports are also I-shaped and attached to vertical web stiffeners by bolts. The stiffeners, 300 x 25 mm² are in turn welded to the web and the flanges of the main girders by fillet welds of width $w = 5$ mm.

As the vertical stiffeners must withstand bearing pressure, there is one additional stiffener on the outside of the main girder.

The cross girders are designed to withstand lateral forces as for example wind and to transfer those forces to the bearings.

The governing load case in the Ultimate Limit State is a future exchange of the pot bearings.
3.1.1 Traffic situation
As previously stated, the Vårby Bridge is consisting of two adjacent composite bridges hosting traffic in opposite directions.

To be able to distinguish between the four main girders, the two girders at the south bridge are in this thesis denoted “A” and “B” and the two the main girders at the north bridge are denoted “C” and “D”. The designations are indicated in figure 18. The monitoring described in chapter 4 was performed at the North Bridge, i.e. main girder C & D, figure 19.

The traffic is by Swedish standards intense as each bridge carries approximately 40 000 vehicles daily.
3.2 Observed cracks

Approximately as many cracks were found at each of the north and the south bridge in 2006. 21 cracks were found at the north bridge and 19 cracks at the south bridge.

Table 1 illustrates the number of cracks observed in [1] i.e. the cracks found 2006.

<table>
<thead>
<tr>
<th>Height of cross girder [mm]</th>
<th>Girder A</th>
<th>Girder B</th>
<th>Girder C</th>
<th>Girder D</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>South bridge</td>
<td>South bridge</td>
<td>North bridge</td>
<td>North bridge</td>
</tr>
<tr>
<td>738</td>
<td>6</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>568</td>
<td>4</td>
<td>2</td>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td>398</td>
<td>1</td>
<td>3</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>Sum</td>
<td>11</td>
<td>8</td>
<td>8</td>
<td>13</td>
</tr>
</tbody>
</table>

Also with respect to the distribution to each of the four individual main girders the number of cracks was found to be of the same order. A few more cracks were found at stiffeners attached to main girders A and D, i.e. the main girders closest to the lanes holding the heaviest vehicles.

Two observed cracks are seen in figure 20. In the picture to the left, one of the “original” cracks detected in the year of 2006 is seen. Prior to the installation of the monitoring equipment (2009), a more thorough inspection of one span, span 5-6 at the south bridge was performed. At that particular span, two cracks were observed 2006.

This time liquid penetrate testing was used and in addition to the original two cracks, another five and previously not detected cracks were found. One of the newfound cracks is seen in figure 20, the picture to the right.

Figure 20 Two examples of cracked welds (toe cracks). Picture to the left is taken from [1]
The deformation field around the tip of any crack can be decomposed into three different basic modes, figure 21, where *Mode I* cracking by far is the most common type of crack mode in cases related to engineering design.

Whereas the cracks occurring at the Vårby Bridge is mode I cracks it should be remembered that during the design phase, only shear forces parallel to the longitudinal axis of the weld were considered. A failure due to such loading would cause a mode II crack.

A short and general description of all three crack modes is given below;

- **Mode I** – The opening mode with a tensile stress normal to the plane of the crack. The crack surfaces moves directly apart
- **Mode II** – The sliding mode with shear stress acting parallel to the plane of the crack and perpendicular to the crack front. The crack surfaces slide over one another in a direction perpendicular to the leading edge of the crack front
- **Mode III** – The tearing mode with a shear stress acting parallel to the plane of the crack and parallel to the crack front. The crack surfaces move relative to one another and parallel to the leading edge of the crack front

### 3.2.1 On-going crack growth process

The propagation rate of existing cracks has been checked once. The measurement of crack lengths was performed by “Projektengagemang i Stockholm AB” and the result of the measurement is reported in [20].

In [20], the length of a crack is defined as in the figure below;

As the author of this thesis defines the length of a crack a somewhat different, the lengths given in table 2 to 4, have been evaluated as \( 0.5 \times (L - t) \), where \( L \) is the length of cracks as given in [20] and \( t \) is the thickness of the stiffeners.
In some cases, lengths in [20] are reported as being less than the thickness of stiffeners (which all are 20 mm thick). It is then, as all cracks seem to initiate on the short side of the stiffeners, assumed that the crack is visible only on the short side of the stiffener and the length, as defined in this thesis, have been assumed to be 1 mm.

To summarize the above discussion, the lengths given in table 2 to 4 differs from the corresponding lengths in [20] and are therefore not to 100 % trustworthy.

Table 2 Length of cracks [mm]. Measurement performed 2010-06-22

<table>
<thead>
<tr>
<th>Girder</th>
<th>Girder A</th>
<th>Girder B</th>
<th>Girder C</th>
<th>Girder D</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:1</td>
<td>Not accessible</td>
<td>Not accessible</td>
<td>Not accessible</td>
<td>Not accessible</td>
</tr>
<tr>
<td>1:2</td>
<td>Not accessible</td>
<td>Not accessible</td>
<td>Not accessible</td>
<td>Not accessible</td>
</tr>
<tr>
<td>1:3</td>
<td>No crack</td>
<td>No crack</td>
<td>9</td>
<td>11</td>
</tr>
<tr>
<td>1:4</td>
<td>10</td>
<td>8</td>
<td>1</td>
<td>10</td>
</tr>
<tr>
<td>1:5</td>
<td>4</td>
<td>1</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>2:1</td>
<td>No crack</td>
<td>No crack</td>
<td>Not accessible</td>
<td>Not accessible</td>
</tr>
</tbody>
</table>

Approximately six months later, the lengths of the same cracks were measured again, see table 3 below;

Table 3 Length of cracks [mm]. Measurement performed 2010-11-30

<table>
<thead>
<tr>
<th>Girder</th>
<th>Girder A</th>
<th>Girder B</th>
<th>Girder C</th>
<th>Girder D</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:1</td>
<td>Not accessible</td>
<td>Not accessible</td>
<td>Not accessible</td>
<td>Not accessible</td>
</tr>
<tr>
<td>1:2</td>
<td>Not accessible</td>
<td>Not accessible</td>
<td>Not accessible</td>
<td>Not accessible</td>
</tr>
<tr>
<td>1:3</td>
<td>No crack</td>
<td>1*</td>
<td>14</td>
<td>12</td>
</tr>
<tr>
<td>1:4</td>
<td>10</td>
<td>14</td>
<td>1</td>
<td>10</td>
</tr>
<tr>
<td>1:5</td>
<td>8</td>
<td>2</td>
<td>13</td>
<td>1</td>
</tr>
<tr>
<td>2:1</td>
<td>No crack</td>
<td>No crack</td>
<td>Not accessible</td>
<td>Not accessible</td>
</tr>
</tbody>
</table>

* At the measurement performed 2010-11-30, a new crack was found at cross girder 1:3, at a stiffener connected to main girder B

The crack growth is then easy to evaluate and is seen in table 4 below;

Table 4 Crack growth [mm] between the 22nd of June 2010 and the 30th of November 2010

<table>
<thead>
<tr>
<th>Girder</th>
<th>Girder A</th>
<th>Girder B</th>
<th>Girder C</th>
<th>Girder D</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1:2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1:3</td>
<td>-</td>
<td>1</td>
<td>5</td>
<td>1</td>
</tr>
<tr>
<td>1:4</td>
<td>0</td>
<td>6</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1:5</td>
<td>4</td>
<td>1</td>
<td>10</td>
<td>0</td>
</tr>
<tr>
<td>2:1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
3.3 Possible causes of the cracks
To be able to determine the cause of the cracks found at the Vårby Bridge and to ensure a (possible) sustainable refurbishment, it is of greatest importance to understand the mechanism behind the cracks.

A few possible causes, more or less likely, were identified and are discussed in this thesis. The possible causes are as follows;

1. Fatigue cracking related to imposed bending of stiffeners and related to the deflection of the concrete deck for heavy vehicles
2. Fatigue cracking related to imposed bending of stiffeners deduced to a differential displacement of the main girders
3. Fatigue cracking due to global bending stress range in the main girder flanges
4. Cracking of the welds due to the shrinkage of the concrete deck.

Though alternative 1 immediately was suspected to be the cause of the cracks, the other alternatives must first be excluded.

3.3.1 Imposed rotation of the stiffeners due to the deflection of the deck
The most likely cause for the observed cracks was immediately believed to be a result of a secondary effect upon the steel as the concrete deck, due to traffic loading, rotates above the main steel girders.

In-between cross girders, the main girder top flanges and web plates provide none or at least very little resistance against the rotation of the deck. The deck can therefore be idealized as being simply supported on the two main girders. This approach has historically been a common practice among bridge designers.

It is however evident that at every cross girder connection, the rotation of the deck will inevitably experience some amount of resistance. The magnitude of the resistance is dependent on adjoining bracing members and the stiffness of those members.

The Eurocode says little in the matter however part 4-2. § 6.6.1.1 (13) states;

"Adjacent to cross frames and vertical web stiffeners, and for composite box girders; the effect of bending moments at the steel-concrete interface about an axis parallel to the axis of the steel beam and caused by deformation of the slab or the steel member should be considered "

The statement is advisory and any further guidance or/and description of methods are not given.
The Occurrence of Cracks at the Vårby Bridge

Figure 23 illustrates the interaction between the deck and the main girders as the deck rotates.

When a heavy vehicle pass by at the cantilever part of the deck, and if shear connectors are near or even in alignment with the vertical stiffener, the hogging bending moment will cause a tensile force to be transferred by shear connectors through the particular weld between the stiffener and the flange, and further into the vertical stiffener. See Case (a) in figure 23.

When a heavy vehicle instead pass by in-between the two main girders, a rotation of opposite sign are produced why instead compressive forces will act upon the stiffener and the weld. See Case (b) in figure 23.

It is not difficult to understand that a cyclic tensile force transferred to the stiffener through the weld could eventually lead to a fatigue crack in the weld of question.

It is however more difficult to understand how a cyclic compressive force could lead to a fatigue crack as such normally doesn’t form in a compressive stress field. To do so, we must introduce what is commonly known as residual stress, which forms during welding.
3.3.1.1 Some parameters of influence

During the welding process, the local heating of the base metal is strongly transient and due to the local nature of the heat source, the base metal temperature varies strongly with the distance from the weld. As the thermal shrinkage of the base metal upon cooling is not uniform, so called residual stress will develop. Residual stress caused by welding have various (and mostly negative) effects upon welded structures. One of the most important effects is an increased susceptibility for fatigue damage.

Barsoum (2008) performed numerical simulations of residual stresses in T-shaped fillet weld joints and compared the result with measured residual stresses. The measurement of residual stress was performed by X-ray diffraction technique.

The numeric result which was found to be in good agreement with the measured result, showed tensile residual stresses in the order of 200 MPa at weld toes and residual stress of the same magnitude but of opposite sign at weld roots. See figure 24 below.

![Figure 24 Simulated residual stresses in T-shaped fillet weld joints. Figures taken from [10]](image)

In this context we must precede matters again and consider a few observations that are discussed later in this report:

- The vast majority fatigue relevant stress at particular areas of interest were found to be compressive, section 4.3
The cracks at the Vårby Bridge originate from toes of the welds and not the roots. See for example the pictures of cracks in figure 20.

Considering the above statements, and the fact that fatigue cracks forms and grow in areas where the resultant stress is tensile, it is reasonable to assume that the residual stress at the weld toes of the particular welds is tensile.

The tensile residual stress is believed to be superposed by cyclic compressive stresses arising in the welds between stiffeners and flanges as heavy vehicles pass by. The result is an effective stress range completely on the tension side and sufficient enough to develop fatigue cracks in a detail subjected to merely nominal compressive loading.

Having established that compressive nominal stresses can cause fatigue cracks, it is of importance to identify other critical parameters for the formation of a fatigue crack and measures taken to identify those. A few of such critical factors are described in the following.

First of all, in welded structures there always exist microscopic flaws or defects acting as stress raisers, amplifying the applied stress. A fatigue crack, as a rule, always initiates at some kind of material defect or flaw. The flaws can be internal, as e.g. a gas inclusion or a cold lap, i.e. a previously just partly melted part of material inside a weld. Flaws are also external, like a mechanical mark or scratch. Parallel to the monitoring of the bridge, a sample of a weld containing a crack was extracted and examined at Luleå University of Technology. The examination is thoroughly described in [6], [7] and [8] and a summary of the examination is found in section 4.4. As no internal flaws were found in the base or weld metal, the particular crack examined was believed to have originated due to a surface defect in a region of high stress concentration.

Secondly, there must exist a fatigue relevant loading pattern containing minimum and maximum stress peak values \((\sigma_{\text{max}}, \sigma_{\text{min}})\) and with some certain variation appearing a sufficient number of times. The stress peaks levels must also be critically different as the algebraic difference between the maximum and the minimum peak governs the nominal normal stress range \((\Delta \sigma = \sigma_{\text{max}} - \sigma_{\text{min}})\) in the crack plane. The stress range is the most important load parameter for the initiation and growth of a fatigue crack. If the stress range is too small, crack initiation will not occur. Due to the performed long term measurements, the magnitude of fatigue relevant stress ranges at the Vårby Bridge are known.
Thirdly, the material need be exposed to a sufficiently large number of cycles of the applied stress. The specific number of cycles required to initiate and grow a crack is also largely dependent on the loading pattern. Also, due to the long term measurements, the specific numbers of fatigue relevant stress ranges at the Vårby Bridge are known.

3.3.2 Imposed rotation of the stiffeners due to a differential deflection of the main girders
Fatigue due to a differential deflection of adjacent main girders is usually associated with so called out-of-plane bending of the main girder web plates, also entitled web-gap bending.

In numerous of bridges all over the world, gaps are present due to the fact that bridge designers previously and in order to avoid welding transverse to tension flanges, excluded weld connections between stiffeners and main girder flanges.

But due to the gap, localized regions of distortional stress occurred when a short unstiffened portion of a main girder web-plate had to accommodate the majority of a cross girder movement due to the differential deflection, see the figure below.

*Figure 26 Differential deflections. Figure taken from [5]*
The prospective problem can be explained by treating the web-gap as a fixed – fixed beam subjected to a rotation at one end.

![Diagram of a cross girder with rotation](image)

Figure 27 Rotation of a cross girder. Figure taken from [5]

The slope equation can be used to find the moment and the maximum stress at the stiffener ends of the web gap.

\[
\sigma_{\text{gap}} = \frac{M \cdot t_w}{2 \cdot I} \quad \text{(Eq. 1)}
\]

\[
M = \frac{2EI}{g} \cdot (2\theta) \quad \text{(Eq. 2)}
\]

Where \( \theta = \frac{\Delta_{\text{girder}}}{L} \) and \( g \) is the height of the web-gap (figure 26).

When combining the two above equations (Eq. 1 & 2) we arrive at a simple expression predicting the web gap stress

\[
\sigma_{\text{gap}} = \frac{2E \cdot t_w}{g} \cdot \left( \frac{\Delta_{\text{girder}}}{L} \right) \quad \text{(Eq. 3)}
\]

In the Vårby Bridge case - if the cracks are allowed to penetrate the full length of the weld between the stiffener and the top flanges - the gap would in this case be the cope holes of radius 50 mm and located between the top flange and the web of the main girder.

From the above expression (Eq. 3) it is seen that a relatively small deflection \( \Delta \) might cause a considerable bending stress if the height of the gap, as in the Vårby case, is small. If reasonable values are substituted into the above expression we get:

\[
E = 210 \text{ GPa}
\]

\[
t_w = 20 \text{ mm}
\]

\[
\Delta_{\text{girder}} = 5 \text{ mm}
\]
The Occurrence of Cracks at the Vårby Bridge

\[ g = 50 \text{ mm (gap = height of existing cope holes at every stiffener)} \]
\[ L = 7500 \text{ mm} \]
\[ \sigma_{gap} = 112 \text{ MPa} \]

It is not hard to predict that a cyclic stress of this magnitude eventually could lead to a propagating crack in the main girder web, with a loss of load carrying capacity as a result. It is thus evident that the main girder web crack is significantly more dangerous for the superstructure as a whole, than the stiffener-to-top flange crack.

As previously stated, the secondary phenomenon of out-of-plane bending of web plates is a well-known problem around the world and occurs particularly in older steel bridges where the vertical stiffeners are not connected (welded) to the tension flange of the main girder.

One famous (or in-famous) Swedish example of out-of-plane bending of the web is the Söderström Bridge in Stockholm with numerous cracks as a result. One such crack is indicated in the figure below and it is worth to notice the orientation of the crack.

Near the stiffener, distortional forces are large and stresses in the vertical direction are accordingly dominating. As cracks predominantly forms perpendicular to a stress field, the crack orientation close to the stiffener thus becomes horizontal. But further away the distortional stresses vanish and the ordinary girder bending stresses starts to dictate the direction of crack growth.

This change of governing stress distribution is the explanation to why web-gap cracks often are seen to change direction when propagating away from the origin, leading to the characteristic “moustache” like form.
3.3.3 Cracking due to global stress in top flanges of the main girders

The connection stiffener-to-flanges is often a governing detail in the FLS (Fatigue Limit State), with a FAT number 71 according to EC 1993-1-9:2005. The FAT number refers to the fatigue strength [MPa] at 2·10⁶ cycles. However, in this particular case, fatigue cracking due to the global bending stress in the top flange can be excluded merely based upon observations of the locations and geometries of the cracks.

The algebraic difference between the tensile and the compressive loads, here represented by the bending moment range (ΔM = M_max - M_min) governs the nominal normal stress range Δσ in the main girder flanges. The stress range is one of the most critical parameter for fatigue failure.

As a majority of the observed cracks at the Vårby Bridge are located in mid span where we - at least for the 1st part of the stress cycle (the sagging bending moment M_max) - can account for a full composite action between the concrete and the steel girders, the compressive stress in the top flange will be reduced to a minimum when considering the fact that the position of the neutral axis of the composite section is located close to the top surface of the steel. At least the absolute value of the compressive stress in the top flanges will fall below the corresponding tensile stress at the bottom flange of the main girder.

We can therefore conclude that the 1st part of the stress cycle (for a sagging bending moment M_max) will affect the bottom flanges more than the top flanges.

As for the 2nd part of the stress cycle (the hogging bending moment M_min), we can only account for a composite action between the reinforcement and the steel girders.
as cracks in the concrete will appear if the bending stress in the deck exceeds the tensile strength of the concrete (normal case). Assuming a reasonable amount of reinforcement, 1 \% of the sectional area of the deck, the bending resistance is approximately the same for the top- and the bottom flange implying numerically equal stresses (but of opposite sign).

The conclusion is thus that the 2\textsuperscript{nd} part of the stress cycle (for a hogging bending moment \(M_{\text{min}}\)) will affect the bottom flange at least equally as much as it will affect the top flange.

It then follows from the above arguing that if any cracking should emerge due to the global bending stress at the stiffener-to-flange connections in a mid-span of a bridge; the cracks should appear at stiffener-to-bottom flange connections and not at stiffener-to-top flange connections.

Moreover, cracks caused by the global stress range in a flange would initiate at the weld toe, but propagate in the plate material. The direction of the crack propagation would be as is indicated in figure 29, i.e. a totally different direction compared to the observed cracks at the Vårby Bridge.

Cracking due to the global bending stress range in the top flanges can consequently be excluded as a probable cause of the observed cracks.
3.3.2 Cracking due to the shrinkage of the concrete deck
Another proposed explanation for the observed cracking is shear failure of the welds due to the shrinkage of the concrete deck.

3.3.2.1 Shear stress upon the welds due to shrinkage
In between the cross girders, the shrinkage is almost unrestrained. However, at a cross girder, some amount of resistance to the contraction of the deck do exist. The result of the resistance are compressive axial forces in the cross girders, introduced through the welds between the vertical stiffeners and the top flanges of the main girder and causing permanent shear stresses acting on the welds. The bracing effect is illustrated in the figure below taken from the FE-model described in Chapter 4. To illustrate the deformation of the steel, the concrete deck has been excluded in the deformation plot.

![Deformation of a cross girder due to the lateral shrinkage of the concrete deck](image)

If the rigidity of the cross girder and the vertical web stiffeners were of infinite magnitude, the contraction of the steel would be of the same order as the shrinkage of the deck, causing a considerable compressive force in the cross girder.

If we for simplicity assume a reasonable contraction of the deck to be in the order of 0.20 ‰, the compressive axial force, $N_{\text{shrink}}$, in a typical intermediate cross girder at the Vårby Bridge would be:

$$
\varepsilon_{\text{shrink}} = -0.002
$$

$$
A = 65\,280\,\text{mm}^2
$$

$$
N_{\text{shrink}} = \varepsilon_{\text{shrink}} \cdot EA = -684\,\text{kN} \quad (\text{Eq. 4})
$$

The above force corresponds to shear stress $\tau_{\text{shrink}}$ along the weld between the stiffeners and the top flange equal to:
The Occurrence of Cracks at the Vårby Bridge

\begin{equation}
\tau_{\\text{shrink}} = \frac{N_{\text{shrink}}}{L \cdot w} = \frac{684\ 000}{400 \cdot 5} = 342\ \text{MPa}
\end{equation}

(Eq. 5)

In the above expression, \( L \) is the total length of the weld and \( w \) is the design width of the weld.

It is evident that the welds wouldn’t be able to resist a shear stress of such a magnitude why a shear failure of the welds would be inevitable.

However, as the steel in real is flexible, the axial force is much smaller, yielding shear stresses along the welds not anywhere near the ultimate shear stress capacity.

In the FE-model and for shrinkage of 0.20 \( \% \), the average normal stress in the cross girder was found to be approximately 4.5 MPa. The stress corresponds to an axial force in the particular cross girder of a magnitude of 73 KN.

The axial force yields a shear stress along the welds between the stiffener and the top flanges of a much more lenient magnitude:

\begin{equation}
\tau_{\\text{shrink}} = \frac{N_{\text{shrink}}}{L \cdot w} = \frac{73\ 000}{400 \cdot 5} = 37\ \text{MPa}
\end{equation}

(Eq. 6)

It’s understood that the magnitude of the above shear stress constitutes no problem as the static shear stress capacity should more than five times higher.

3.3.2.2 Normal stress perpendicular the welds due to shrinkage

However, the shrinkage of the concrete deck might have other substantial effects upon the welds of interest.

When the concrete, due to the shrinkage, contracts a certain distance \( \Delta \), the steel beneath is bound to follow the contraction.

As mentioned earlier, in between the cross girders the shrinkage is almost unrestrained. However, at stiffeners and cross girders and due to the stiffness and the bracing of those members, some amount of resistance against the contraction exists.

As the deformation of the deck arise due to restraint and not from any real loading, the part of the deck between the two main girders will - figuratively speaking - act as a “rigid spring” counteracting its own flexure.

The effect is visualized in the figures below where it is seen that the part of the deck between the girders is almost horizontal.
The response of the cross girder and particularly the web stiffener is highly dependent on the presence of shear connectors as such are the only members that are able to transfer a tensile force between the concrete and the steel.

As shear connectors normally are welded to bridge main girders without any consideration to positions of web stiffeners, we must consider two equally as probable cases;

- Case A - shear connectors near or in alignment with a vertical stiffener
- Case B - no shear connector near the vertical stiffener

**Case A - Shear connectors very near or in direct alignment with a vertical stiffener**

In this case and due to the flexure counteracting, the shear connector will elongate when being subjected to a tensile force.

The force will be further transferred from the shear connector into the stiffener, causing a local tensile peak stress, $\sigma_{\text{stiff, shrink}}$ in the stiffener.

Yet again, if the rigidity of the cross girder and the vertical stiffener is large, the force in the shear connector can be estimated by simple means of geometry.

Assuming again $\varepsilon_{\text{shrink}} = 0.20 \%$, the deck will contract a distance $\Delta = 0.5 \cdot B \cdot \varepsilon_{\text{shrink}}$, where $B$ is the width of the part of the deck between the two main girders.
The occurrence of cracks at the Vårby Bridge

Figure 35

Figure 36

The width of the gap is assumed to correspond to the elongation of the shear connector and can be evaluated as:

\[ g = \frac{\varepsilon \cdot 0.5 \cdot B}{h} \]

In the above expression, \( b_{fl} \) is the width of the top flange and \( h \) is the distance between the top surface of the cross girder and the centre of gravity of the deck.

\[ g = \frac{0.0002 \cdot 0.5^2 \cdot 700 \cdot 7500}{550} = 0.48 \text{ mm} \]  

(Eq. 8)

At the Vårby Bridge, the position of the shear connectors is 75 mm from the edge of the flange. Thus the elongation of the shear connector can be approximated as 21% lesser than the gap, i.e. \( (1 - 0.21) \cdot 0.48 \text{ mm} = 0.38 \text{ mm} \).

An elongation \( \Delta L = 0.38 \text{ mm} \) of a shear connector with a diameter of 22 mm and length 175 mm and in alignment with the stiffener implies a tensile normal force \( N_{axial} \) in the order of:

\[ N_{axial} = \frac{0.38 \cdot 210 \cdot \pi \cdot 11^2}{175} = 173 \text{ KN} \]  

(Eq. 9)

The above normal force corresponds to a stress in the shear connector beyond the yield limit of the steel or a pull-out of the shear connector from the concrete and is not realistic but for the sake of the example that is for the moment overlooked.

Assuming further that the axial force in the shear connector is transferred into the stiffener and that the force spreads through the top flange of the main girder, the peak stress in the stiffener due to the shrinkage of the concrete deck can be estimated as:
\[ \sigma_{\text{stiff, shrink}} = \frac{N_{\text{stud}}}{20 \cdot (22 + 2 \cdot 20)} = 140 \text{ MPa} \]  
(Eq. 10)

As concluded earlier in this section, the steel cross girder is not rigid but flexible why the above peak stress by all reasoning should be overestimated.

In the finite element model used for the purpose of this thesis, the effect of shrinkage upon the steel cross girders was simulated. To make the connection between the steel and the deck as rigid as possible, a shear connector was placed directly in alignment of the vertical stiffeners.

![Figure 37 Stiffener stress due to lateral shrinkage of the concrete deck](image)

The tensile stress peak in the outmost top corner of the stiffener in the FE-model was found to be 52 MPa which is about 37 % of the analytic value of \( \sigma_{\text{stiff, shrink}} \). The axial force in the shear connector was 56 KN, i.e. approximately 33 % of the analytic value of \( N_{\text{axial}} \).

Even if the stress peak found in the finite element model is moderate in comparison to the equivalent analytic stress peak (140 MPa), it is important to recognize that it in fact correspond to a stress \( \sigma_{\text{shrink}} \) in the welds of interest of a magnitude as high as > 100 MPa when regarding the fact that the width of the fillet welds at the Vårby Bridge are half of the width of the stiffener. Moreover, the stress is permanent and the direction of the stress field is normal to the plane of the cracks found at the Vårby Bridge.

As was concluded in section 3.3.1, such a tensile and steady state stress will by all reasoning affect the fatigue resistance negatively as it along with residual stresses raises the fatigue cycle mean stress in the weld. Furthermore, once a fatigue crack is formed, the stress would undoubtedly affect the stress state around tip of the crack in a negative way.
The Occurrence of Cracks at the Vårby Bridge

If we for the time being only consider the static effect of the lateral shrinkage upon the welds of interest, a shear stress $\tau_{l,\text{shrink}}$ along the weld and a tensile normal stress $\sigma_{\perp,\text{shrink}}$ perpendicular the weld have been identified.

For reasons of simplicity, the normal stress $\sigma_{\perp,\text{shrink}}$ acting on the weld is conservatively assumed to be twice the normal stress $\sigma_{\text{stiff, shrink}}$ which is acting on the stiffener.

$$\tau_{l,\text{shrink}} = 37 \, \text{MPa}$$

$$\sigma_{\perp,\text{shrink}} = \tau_{\perp,\text{shrink}} = \frac{2 \cdot 52}{\sqrt{2}} = 74 \, \text{MPa} \quad (\text{Eq. 11})$$

According to EC 1993-1-8:2005, the resistance of a weld can be verified by the following expression:

$$\sqrt{\sigma^2 + 3(\tau^2 + \tau^2)} \leq \frac{f_u}{\beta_w \cdot Y_{M2}} \quad (\text{Eq. 12})$$

$$\sqrt{(74)^2 + 3 \cdot (37^2 + 74^2)} = 160 \, \text{MPa} \ll \frac{f_u}{\beta_w \cdot Y_{M2}} \quad (\text{Eq. 13})$$

One conclusion is thus that the combined effect of stress perpendicular the weld and shear stress along the weld and due to shrinkage of the concrete deck can be excluded as a reason for the failure of the welds.

Another, more important conclusion is that the shrinkage of the concrete deck possibly can cause a permanent tensile stress in stiffeners regarded a presence of shear connectors near or in alignment of vertical stiffeners.
Case B - No shear connector near the vertical stiffener
The next question is; what will happen if there are no shear connectors directly in the vicinity of the vertical stiffener?

Let's assume a gap between the deck and the top flange and of a width, 35 % of the analytic value of \( g \) derived earlier.

The assumed value of 35 % corresponds to the elongation of a shear connector situated directly on top of the vertical stiffener in the FE-model.

\[
\Delta L = 0.35 \cdot 0.48 = 0.17 \text{ mm} \quad \text{(Eq. 14)}
\]

As the width of the top flange, \( b_{fl} \) is 700 mm; the rotation \( \theta \) of the stiffener is given as;

\[
\theta = \frac{\Delta L}{0.5 \cdot b_{fl}} = \frac{0.17}{350} = 0.00049
\quad \text{(Eq. 15)}
\]

The above rotation is as an example numerically much larger than the analytic rotation at mid support, caused by the vehicle with known weight and position and as described in section 4.2.1.

The fact that a permanent rotation caused by the shrinkage could be larger than rotations caused by heavy vehicles means that the possible gap between the deck and the top flange will not close entirely when the deck rotates for the heavy vehicle. This in turn means that the compression part of the forced couple and due to the bending moment will be transferred further towards the centreline of the main girder. See figure 38;

![Figure 38](image)

**Figure 38 Transfer of compressive force towards the centre of the main girder**

As a consequence, the compressive stress in the outmost top corner of the stiffener will be reduced.
Hints of such gaps between the concrete deck and the steel top surface at positions of cross girders are observed at for instance the Vårby Bridge, see the figures below;

![Figure 39 Hints of gaps at intermediate cross girders. The Vårby Bridge](image)

At some connections traces of corrosion are seen as water and oxygen have access to the on the top surface, un-painted flanges.

The depth, width and length of gaps at four different intermediate cross girders have been measured in the spring of 2012.
The measurement took place in span 1-2 at the south bridge. Unfortunately, the measurement was performed at intermediate cross girders only but it should be noted that the discussed possible effect by all reasoning should be more prominent at mid supports where the much stiffer cross girders also rest on bearings and consequently are more restrained to “move along” with the deck.

The result of the measurement is seen in the table below;

<table>
<thead>
<tr>
<th>Height</th>
<th>Girder</th>
<th>Depth</th>
<th>Width</th>
<th>Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1</td>
<td>A</td>
<td>568</td>
<td>0.15</td>
<td>100</td>
</tr>
<tr>
<td>F1</td>
<td>B</td>
<td>100</td>
<td>0.15</td>
<td>50</td>
</tr>
<tr>
<td>F2</td>
<td>A</td>
<td>398</td>
<td>0.15</td>
<td>100</td>
</tr>
<tr>
<td>F2</td>
<td>B</td>
<td>40</td>
<td>0.15</td>
<td>100</td>
</tr>
<tr>
<td>F3</td>
<td>A</td>
<td>568</td>
<td>0.15</td>
<td>30</td>
</tr>
<tr>
<td>F3</td>
<td>B</td>
<td>10</td>
<td>0.15</td>
<td>100</td>
</tr>
<tr>
<td>F4</td>
<td>A</td>
<td>738</td>
<td>0.15</td>
<td>40</td>
</tr>
<tr>
<td>F4</td>
<td>B</td>
<td>60</td>
<td>0.15</td>
<td>400</td>
</tr>
</tbody>
</table>

The most interesting observations are listed below;

1. The gaps were found to be narrow, in the order of 0.2 millimetres. The order corresponds fairly well with what’s to expect from rotations due to shrinkage as described previously in this section.

2. All gaps were found at the inside of the main girders and none were noted at the outside of the main girder web plate. This observation was also expected. Since the cantilevers of the deck are not restrained and consequently are free to deform, no gaps should occur between the deck and top flange at positions on the outside of the main girder web plate.

3. The gaps with largest depths occurred most frequently at the main girder denoted A i.e. the main girder near the lane hosting the heaviest vehicles.

One other example of a bridge with observed gaps is Bridge 13-803-1(2), located in Halmstad, Sweden. A bridge quite similar to the Vårby Bridge with a slender deck and rigid web stiffeners and cross girders.

One distinct difference is that the lateral spacing between the shear connectors are much narrower at the bridge in Halmstad.
The occurrence of cracks at the Vårby Bridge

At one mid support, the width of the gap was measured as wide as 6 mm. The depth of the gap towards the centre of the main girder was found to be 14 cm and at another support 20 cm. At a distance before and after the support, the gap vanished. Indications of gaps were also found at intermediate cross girders but as distinct as at the supports.

It is unquestionably so that the shrinkage of the concrete deck alone could not cause gaps as wide as 6 mm. However, it can’t be out ruled that gaps caused by the shrinkage, but of a much smaller magnitude, could have provided access to water to infiltrate the interstice between the flange and the deck. Where there is access to water and a restriction against volumetric expansion, distress to the concrete could have commenced from the first freeze/thaw cycle and continued throughout successive winter seasons, eventually resulting in a loss of concrete surface.
3.3.2.3 Discussions concerning the effect of shrinkage upon vertical stiffeners and cross girders at the Vårby Bridge

Given shear connectors near or in alignment of vertical stiffeners, there could be a possibility that tensile forces (due to shrinkage) can be introduced into stiffeners by way of shear connectors. The forces would then cause a permanent tensile stress field perpendicular to the welds of interest. Even if the magnitude of the stress isn’t sufficient enough to explain the cracks, the direction of the stress field would be normal to the planes of the observed cracks. Such a tensile, steady state stress would affect the fatigue resistance of the particular welds negatively as it raises the mean stress of fatigue stress cycles. Furthermore, once a fatigue crack is formed, such stress would undoubtedly affect the stress state around the tip of the crack in a negative way.

Given no shear connectors near vertical stiffeners, the shrinkage could possibly cause gaps between the deck and the steel top flange. Such gaps are found at cross girder connections at the Vårby Bridge and at other comparable bridges. In case of a heavy vehicle positioned in-between the main girders, causing nominal compressive stress in stiffeners as described in section 3.3.1 case b, such gaps might in fact reduce the compressive stress peaks at the outmost top corners of stiffeners. The peak stresses are instead transferred towards the centre of the main girders, i.e. closer to the main girder web plates. The effect could possibly be even more prominent at supports and for the following reasons;

- The cross girders are at mid-supports much stiffer in comparison to the intermediate cross girders
- As the cross girders stands on bearings, they are more reluctant to bend as the deck rotate
3.4 Observations of similar cracks in other countries

Similar damages as those observed at the Vårby Bridge have also been observed in other countries. The figure below shows examples of cracked welds between vertical web-stiffeners and steel top flanges in Austrian bridges. An investigation [2] under the auspices of ASFINAG - the Austrian highway authorities - has been conducted by Professor Richard Greiner et al, at Graz University of Technology.

![Figure 42 Observed cracks at an Austrian composite bridge][2]

Cracks were found at number of composite highway bridges and the investigation made clear that the lifetime of the weld connection is influenced by numerous structural effects, such as:

- the bending stiffness of the concrete slab
- the thickness of the upper flange
- the bracing system of the cross-section

Moreover, intense stress concentrations at the outermost edges of the stiffeners were found.

The investigation focussed on a loading condition with the load located at the cantilever of the deck but the investigation also states that vehicles located centrally between the two main girders yield fatigue relevant stresses in the studied connections of a comparable magnitude.

Some conclusions made by the Austrian investigators were that high fatigue relevant stresses in highway composite girder bridges with vertically stiffened web plates can be caused by the deformability of the concrete slab and the restraining effects of adjacent bracing elements.

Furthermore, some advices intended for future designers of composite bridges were given:
First of all, it should be avoided to place horizontal bracing elements vertically close to such connections.

If that can’t be avoided, care should be taken to the following according:

- make the concrete slab sufficiently thick and stiff
- place the shear connectors, between the concrete deck and the main girders, in close transversal spacing in order to avoid the introduction of a large bending moment into the main girder at a stiffener
- Direct the heavy traffic over one lane only and as close as possible to a main girder. Avoid change of sign of the load in the connection
- Full penetration welds should be used instead of fillet welds. Thereby allowing the increase of fatigue strength by a factor of the order of 2, i.e. FAT 71 instead of FAT 36/45
- post-welding improvement methods could additionally be taken into consideration

Cracks have also been observed in a large number of bridges in Japan. Examples are seen in figure 43.

Crack no. 1 occurred at the top of a vertical stiffener welded to the upper flange of the main girder and crack no. 2 at the weld between the web and the top flange of the of the main girder. The latter crack appeared after the first crack (no. 1) had penetrated the top end weld of the vertical stiffener completely.

The investigation recognized the deflection of the deck by heavy traffic as the reason for the 1st crack in figure 43 and the relative deflection of the main girders as the main reason for the 2nd crack. In this context it must be understood that the 2nd crack would not have been formed if the 1st crack hadn’t penetrated the top end weld between the stiffener and the flange.

The solution to the problem was a refurbishment by adding stringers in-between the main girders, thereby reducing the deflection of the deck.
Another Japanese investigation [17] also reports damages (cracks) found between vertical stiffeners and top flanges at a bridge, see figure 44. Dynamic strain measurements using WSN (Wireless Sensor Network) was used together with FE-analyses. The investigation identified a deformation induced fatigue to be the cause of the cracks.

Two types of deformations/deflections were identified:

1. The differential deflection of the main girders and;
2. The transversal deflection of the concrete deck

It was concluded that the above deflections force the vertical stiffeners to bend. But as the upper strut of the cross girders restricts this deformation, high stresses occurred in the top corner of the stiffeners.
4 The Monitoring of the Vårby Bridge

4.1 Bridge monitoring

Extensive field measurements were carried out in the summer of 2009. Strains as well as deflections were measured with strain gauges, inclinometers and LDVT.

In addition a steel sample containing a part of a crack surface was extracted from the bridge.

Two types of measurements and with different purposes were carried out:

1. Measurements conducted for a vehicle of known weight and where speed and position were noted
2. Long-term monitoring to assess effects of daily traffic flows.

In addition to the measurement, a sample of a weld, containing a crack surface was extracted from one stiffener.

4.1.1 Installation of the monitoring equipment

As a first step, all connections of interest between support 5 and 6 of the bridge were examined in a more thorough way.

This time the welds were inspected not only visually but with magnetic crack-detection equipment. To be able to follow the crack growth process, the tip of the most substantial crack was marked in a sustainable way.

The following three unique positions at different cross girders were chosen for the measurements.
The monitoring of the Vårby Bridge

Figure 46 Positions of the measure equipment

- Cross girder denoted F21. Single web stiffeners. Cracks have been found in the welds of interest at both the main girder C and D
- Cross girder denoted F23. Single web stiffeners. No observed cracks
- Cross girder denoted B6 at mid support no. 6. No observed cracks

A total number of 22 strain gauges were installed, distributed in the following way;

- Cross girder F21: Gauge no. 16-23
- Cross girder F23: Gauge no. 1-8, 10, 11
- Cross girder B6: Gauge no. 12-15

A more detailed description of the individual position of each strain gauge can be found in [4].

The majority of all installed strain gauges were for obvious reasons orientated to measure the longitudinal strain of the stiffeners, but some strain gauges (number 9, 10 & 23) were positioned on the top flange of the attaching cross girder.

Gauge no. 4 & 8 were positioned at the bottom surface of the lower flanges. These two gauges (number 4 and 8) were considered as essential as they were the only gauges providing strains of a more global nature.

Of interest was also the absolute rotation of the concrete deck for a given load in certain different positions and the relative difference between the concrete and the steel, i.e. the gap opening up between the deck and the top flange of the steel girders as a vehicle passes by on the cantilever of the deck.
For that purpose, one LDVT and one inclinometer were also installed. The LDVT and the inclinometer, as well as gauge no. 5, 6, 7, 11 and 24 are seen in the picture to the left.

The LDVT was set to measure the distance between the lower surface of the top flange and the corresponding surface of the deck. A decrease of the distance, i.e. when the load is positioned between the two main girders, has in [4] been given a positive value.

4.2 Short term monitoring and numeric analyses
The short term measurements for a vehicle of known parameters like weight, speed and position were carried through in June 2009. As the traffic was intense and the bridge couldn’t be closed, the measurements were performed during night time.

**Vehicle used**
While performing the measurement, the following parameters related to the testing vehicle were considered important:

- Gross weight of the vehicle
- Individual axle/bogie weight
- Basic geometry of the vehicle, i.e. distance between the front and the rear axles, the individual distance between the axles within the bogie and the wheel base
- Position(s) of the vehicle on the bridge.
The monitoring of the Vårby Bridge

Load positions
A total of 21 unique crossings over of the bridge were registered during the night between the 5th and 6th of June, 2009. In the following, two of the passages are described more thoroughly.

The two chosen passages are:

- Passage 016. The vehicle travelling in lane 2 in-between the two main girders
- Passage 018. The vehicle travelling at the cantilever of the deck outside of main girder D

FE-model
The entire structure - the concrete deck as well as all steel plates - was composed by 4-node quadrilateral shell elements with six degrees of freedom in each node. A piece of the model can be seen below.

Figure 49 A piece of the FE-model

Mainly due to the limitations of computer calculation capacity, only two spans of the bridge were modelled. For the same reason as above, the element size of the rectangular mesh in the particular area of interest, the vertical stiffeners, was 50×80 mm² for the single stiffeners attached to intermediate cross girders in and 50×100 mm² for the stiffeners attached to the cross girder at mid support. The model was thereby considered to be inadequate for evaluations of, for example stress concentrations in areas of interest, but still adequate for the purpose of this report. A refinement of the model would be possible, for instance by creating fine-mesh sub-models.

To be able to evaluate the effect of the rotation of the deck upon the cross girders and stiffeners, the top flanges and the concrete deck were tied together by so called
rigid links. Defining a rigid link between nodes is the same as introducing rigid compatibility conditions with respect to selected displacements in these nodes. The top flanges of the main girders are thereby forced to deform in the same manner as the concrete deck.

In areas where tensile forces are expected to be transferred to the steel, rigid links were substituted by beam elements corresponding to the shear connectors on the steel top flanges. The beam elements were given a high bending stiffness but correct axial stiffness.

Figure 50 Connection between the deck and the steel top flange

To avoid a transfer of a tensile force into single discrete points at shell elements; the shear connector was modelled as, not one, but two beam elements. Each element with a cross sectional area of \(0.5 \times A_{\text{shear connector}}\).

As the same model also was used for the evaluation of the effects for a vehicle standing either on the cantilever of the deck or in-between the two main girders; the position of the rigid links and the beam elements corresponding to the shear connectors had to be re-arranged a little depended of the position of the load.

Case A & C:
The vehicle positioned laterally in-between the two main girders (in lane no. 2). Either in a longitudinal position above an intermediate cross girder (case A) or a cross girder at mid-support (case C)

Case B & D:
The vehicle positioned at the cantilever of the concrete deck. Either in a longitudinal position above an intermediate cross girder (case B) or a cross girder at mid-support (case D)

The four different cases (A, B, C & D) are illustrated graphically in the figures below;
Furthermore, to examine effects upon the phenomena discussed in section 3.3.2.2, of gaps reducing compressive peak stresses at top corners of stiffeners, the original finite element model was modified so that a few chosen rigid links were removed.

This was assumed to correspond to gaps with a depth of 150 mm at main girder D and 100 mm at main girder C. Main girder D is the girder near the traffic lane holding the heaviest vehicles.
The vehicle described earlier in this section was modelled at various longitudinal positions as 6 discrete static point loads centred directly above the cross girder of interest.

When evaluating effects of shrinkage, a uniform thermal load of $-20\,^{\circ}\text{C}$ was imposed upon the bridge deck shell elements. As the thermal expansion coefficient $\alpha$ is $1 \cdot 10^{-5}/\,^{\circ}\text{C}$, the load was assumed to correspond to a shrinkage of 0,20 $\%$. 

*Figure 56 Excluded rigid links at a cross girder at mid support. Vehicle in between the main girders*
4.2.1 Passage 016 in between the main girders

In the passage described in this section the vehicle is crossing the bridge in lane no. 2, see figure 57. The particular passage was chosen to illustrate the effect of the rotation of the deck with a vehicle standing in between the two main girders.

As the vehicle is travelling in lane no. 2, i.e. almost centred between the two main girders, the effect of the weight of the vehicle should be small, almost negligible.

In the following, the result of the measurement is compared to results achieved in two separate finite element models. The two models are denoted FE-model 1 and FE-model 2 as described below;

As stated earlier, to examine the phenomena discussed in section 3.3.2.2, of existing gaps between the deck and the flange reducing compressive peak stress peaks in top corner of stiffeners, the original finite element model (FE-model 1) was modified a little, leaving us with two similar but not identical models;

- FE-model 1:
  A complete set of rigid links, i.e. the top flange of the steel main girder will deform exactly as the concrete deck above

- FE-model 2:
  Identical to FE-model 1 but with the following exception. A few chosen rigid links were removed, see figure 56. This was assumed to correspond to a gap of a depth 150 mm (main girder D) and 100 mm (main girder C).

Global behaviour

The diagram in figure 57 illustrates the global bending stress in the main girders as measured with Ch. 4 and Ch. 8, i.e. the strain gauges at the bottom flanges of the two main girders.

The strains and the corresponding stresses from the two gauges describe an almost perfect influence curve illustrating the bending moment distribution in the particular points as the vehicle is crossing the bridge.

As lane number 2 is situated slightly closer to main girder C, we expect and receive a higher stress in Ch. 4
For the main purpose of this report, the stresses representing the global behaviour of the superstructure is of no particular importance but still interesting and essential for a numerical model validation.

Furthermore, as the distance between the two main girders is 7,50 m, the transverse position of the vehicle could in advance be determined to be approximately 3,53 m from main girder C, i.e. a position almost in the centre between the two main girders.

The FE-stresses were found to be slightly higher in comparison to the measured, but the difference was - for the purpose of this report - considered to be within in the range of acceptance.

<table>
<thead>
<tr>
<th>Channel</th>
<th>(\sigma_{\text{Meas}})</th>
<th>(\sigma_{\text{FE}})</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>6,2</td>
<td>8,5</td>
<td>Bottom flange. Main girder C</td>
</tr>
<tr>
<td>8</td>
<td>5,5</td>
<td>7,5</td>
<td>Bottom flange. Main girder D</td>
</tr>
</tbody>
</table>
The monitoring of the Vårby Bridge

**Stresses in the vertical stiffener at a mid-support**

Figure 58 illustrates the state of stress in channel no. 12 & 14, i.e. the cross girder at mid support 6. As the position of the vehicle is almost centred between the two main girders, tension is expected in the stiffener situated on the outside of the web of the main girder (Ch. 14) and compression in the stiffener on the inside of the web (Ch.12).

Intuitively the compressive stress (Ch. 12) was expected to be of same order or more likely higher than the corresponding tensile stress (Ch. 14). This is due to the following reason.

Assume a force couple $\pm F$ due to the hogging bending moment. The compressive part of the force couple is assumed to be distributed to the steel as a linearly varying pressure between the deck and the top flange. The tensile part of the force couple is assumed to be transferred to the stiffeners by way of the shear connectors. Along the way from the shear connectors to the stiffener, some reduction of tensile force should take place and the maximum stiffener stress peak should be reduced. The reduction or redistribution is assumed to exist due to internal flexibilities as for example the deformability of the steel top flange for loading perpendicular the plane of the flanges and the axial stiffness of the shear connectors.

This effect was also observed when evaluating stiffener stresses in FE-model 1, i.e. the model with a complete set of rigid links. However, in the measurement data, the
effect was absent as the compressive stresses at Ch.12 was found to be 52 % (= 14/27) of the corresponding tensile stress at the position of Ch.14.

In the finite element model, the maximum tensile stress in the stiffener located on the outside of the main girder web plate (Ch.14) was found to be in the same order as the measured stress. The compressive stress peak in the stiffener located on the inside of the main girder web plate was found to be of a much higher magnitude in comparison to the measured stress, see the figure to the left.

This behaviour - or discrepancy - was regarded to be in-consistent and difficult to explain.

As faulty strain gauges were not considered to be a trustworthy option, the discrepancy between the FE-model and the measured results had to be found elsewhere.
The discrepancy was instead assumed to be caused by one of the following:

1. An in-accurate FE-model
2. Gaps between the concrete deck and steel top flange transferring the compressive part of the force couple closer to the center of the main girder

As the FE-model was considered to principally represent the behaviour of the real structure accurately, the validity of the FE-model could be checked by evaluating the bending moment or restrain in the FE-model and thereafter judge the reasonability of the bending moment.

It was considered that the response of the was far too rigid, yielding far too high peak stresses, but at least the relative distribution of stress upon each of the two stiffeners was assumed to be correct.

The bending moment of restraint could be evaluated by the integration of stress over the two stiffeners. As the neutral axis of the participating cross section is located in the centre of the main girder web plate, the force carried by the same plate was considered to be of less importance.

The compressive force (per millimeter) at the stiffener on the inside of the main girder web plate was found to be \(-5377 \text{ [MPa*mm]}\) and the corresponding integrated tensile force (per millimeter) at the stiffener on the outside of the main girder web plate was found to be \(3057 \text{ [MPa*mm]}\).

The difference between compressive- and tensile stress, \(2320 \text{ [MPa*mm]}\) i.e. a force \(-25^*2,32 \text{ KN} = -58 \text{ KN}\) was assumed to be the axial force corresponding to the weight of the bogie of the vehicle. The total weight of the bogie is 14,26 tons, i.e. 142,6 KN.

The FE-shell model was constructed in such a way that at a section equal to the upper edge of the stiffener and for a load case with the vehicle standing in-between the two main girders only the stiffener on the inside of the main girder web-plate should carry weight of the vehicle. This is due to the orientation of the so called "rigid links" tying the deck and the steel girders together. It should in this context be noted that above axial force due to the weight of the vehicle corresponds to a compressive stress only around \(-7 \text{ MPa}\) when distributed over one stiffener. If the...
force is assumed to be carried by two stiffeners and some certain length of the main girder web plate, the impact become in principal negligible.

If the axial force is subtracted from the integrated compressive, the bending moment in the stiffener due to the imposed rotation can be evaluated as:

\[ M = 35 \text{ KNm} \]

The above bending moment corresponds to the following analytical stress in the stiffeners:

\[ \sigma = \pm 22 \text{ MPa} \]

The bending moment \( M \) can to some extent be judged by imposing proper rotations upon a beam-element frame. In lack of data from the measurement, the magnitude of the rotations could tentatively be collected from the FE-model. According to the FE-model, the rotation of the deck above main girder C and D are:

\[ \theta_{\text{girder D}} = -0.00020 \]
\[ \theta_{\text{girder C}} = 0.00023 \]

Imposed on a frame according to the figure below, the following bending moments are achieved:

\[ M_{\text{girder D}} \approx 37 \text{ KNm} \]
\[ M_{\text{girder C}} = 42 \text{ KNm} \]

![Figure 62 Bending moment of restrain due to imposed rotations](image)

It is seen that the bending moment \( M_{\text{girder D}} (= 35 \text{ KNm}) \) in the stiffener attached to main girder D and obtained in the FE-shell model comes fairly close to the
corresponding bending moment in the simple beam-element frame (37 KNm). One note is however that the achieved magnitudes of the bending moments were found to be highly depended on adopted boundary conditions between the steel and the concrete. Especially for translations in the horizontal direction.

The 2\textsuperscript{nd} possible reason for the discrepancy is the assumption of gaps between the concrete deck and the steel top flange as discussed in section 3.3.2.3.

It was found that, with gaps, a better concurrence between measured data and finite element results is found. Compare with the figure and the table below:

![Stiffener stresses at mid-support. FE-model 2 with gaps](image)

<table>
<thead>
<tr>
<th>Channel</th>
<th>(\sigma_{\text{Meas}})</th>
<th>(\sigma_{\text{FE-1}})</th>
<th>(\sigma_{\text{FE-2}})</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>-14</td>
<td>-51</td>
<td>-15</td>
<td>Stiffener inside of web plate. Girder D</td>
</tr>
<tr>
<td>14</td>
<td>27</td>
<td>31</td>
<td>36</td>
<td>Stiffener outside of web plate. Girder D</td>
</tr>
</tbody>
</table>

Given the existence of a gap, the compressive stress at the position of Ch. 12 will be reduced as the top corner of the stiffener doesn’t capture the entire rotation of the deck and the stiffener will deform in a manner in accordance with figure 64.

The position of the compressive peak stress is instead shifted, from the top corner to the end of the gap at a distance along the edge of the stiffener and towards the centre of the main girder.
Figure 64 Deformation of the stiffeners with a gap between the concrete deck and the steel top flange

To summarize findings from the comparison of FE-analyses with measured data, the following can be stated:

- The bending moments in the stiffeners in FE-shell element model can be reproduced when rotations from the same analysis are imposed upon a simple beam-element frame.

- Stiffener stresses in the FE-shell element models are satisfactory but only if gaps between the concrete deck and the steel top flange are introduced. Without gaps, the response of the FE-model is too rigid, yielding too high stiffener stresses when compared to measured ones.

- One consequence of such gaps would be a “softer interaction” between the steel and the deck in the sense that the resultant of compressive force is transferred towards the centre of the main girder and the compressive stress at the outmost corner of the stiffeners is reduced.

The described effect should by reasoning be more prominent at mid supports for the following reasons; the cross girders at mid support are significantly stiffer compared to the intermediate cross girders and the cross girders at mid supports are standing on bearings. This means that they are more prohibited to “follow” the rotation of the deck.
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Stresses in vertical stiffeners at intermediate cross girders

Figure 65 illustrates the state of stress in the two vertical stiffeners connected to cross girder F23. Ch. 5 is situated on the stiffener attached to main girder D and Ch.1 on the stiffener attached to the main girder C.

As the vehicle pass by the cross girder, negative strains are registered by both channels. However, it is seen that the stress peak in channel 1 is significantly higher.

Table 8 Stiffener stresses at an intermediate cross girder. Passage 016

<table>
<thead>
<tr>
<th>Channel</th>
<th>( \sigma_{\text{Meas}} )</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-24</td>
<td>Vertical stiffener. Main girder C</td>
</tr>
<tr>
<td>5</td>
<td>-15</td>
<td>Vertical stiffener. Main girder D</td>
</tr>
</tbody>
</table>

The position of the vehicle is shifted towards main girder C, thus a higher rotation and accordingly a higher stress peak was expected at Ch. 1. However, the large difference of stress was surprising. In fact, the difference is difficult to explain, however an attempt is made in this section.

First of all, the rotations achieved in the FE-model are being studied. The shifting of the load towards main girder C means that:

Alignment of the text with the table data and diagram provided.
the rotation of the deck above main girder C should be larger than the corresponding rotation above main girder D

- the deflection of main girder C should be larger than the corresponding deflection of main girder D

According to the FE-analysis, the rotation of the deck above the two main girders are:

<table>
<thead>
<tr>
<th>Girder</th>
<th>$\theta$</th>
<th>$\Delta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>D</td>
<td>0,00036</td>
<td>3,8</td>
</tr>
<tr>
<td>C</td>
<td>-0,00039</td>
<td>4,0</td>
</tr>
</tbody>
</table>

At a first glance, the above rotations also seem surprising as a numerically larger rotation was noted above main girder D, i.e. the main girder furthest away from the load. However, as main girder C is carrying the larger portion of the load, the deflection of the girder C will also be proportionally larger.

The rotation due to the differential deflection will burden the stiffener attached to main girder D and unburden the stiffener attached to main girder C. In principal according to the figure to the left.

One problem is that it isn't possible to distinguish between the two different kinds of rotations in the FE-model.

The rotation due to the differential deflection can however be estimated as:

$$\theta_D = \pm \frac{6 L}{L} = 0,0003$$, where $L = 7,5$ m is the length between the main girders

With an assumption that the rotation due to deflection of the deck to approximately 6% larger above main girder C compared to main girder D, the following aggregate rotations are achieved:
The monitoring of the Vårby Bridge

### Table 10 Aggregate rotation

<table>
<thead>
<tr>
<th></th>
<th>Girder D</th>
<th>Girder C</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\theta_{tot}$</td>
<td>0.00036</td>
<td>-0.00038</td>
</tr>
<tr>
<td>$\theta_A$</td>
<td>0.00003</td>
<td>0.00003</td>
</tr>
<tr>
<td>$\Sigma \theta$</td>
<td>0.00039</td>
<td>-0.00036</td>
</tr>
</tbody>
</table>

It is then seen that even if the position of the vehicle is shifted towards main girder C, it is still possible that the rotation above main girder D is larger when the differential deflection of the main girders are taken into account.

The FE-stresses in the stiffeners were then being studied. In general, the stresses were yet again found to be considerably higher in the finite element model when compared to measured results. In particular in the one finite element model with a complete set of rigid links (no gaps).

Compare with data in table 11 where the FE-stress corresponding to Ch. 1 was found to be -69 MPa and the FE-stress corresponding to Ch. 5 was found to be -67 MPa.

<table>
<thead>
<tr>
<th>Channel</th>
<th>$\sigma_{\text{measured}}$</th>
<th>$\sigma_{\text{FE-1}}$</th>
<th>$\sigma_{\text{FE-2}}$</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-24</td>
<td>-69</td>
<td>-35</td>
<td>Vertical stiffener. Main girder C</td>
</tr>
<tr>
<td>5</td>
<td>-14</td>
<td>-67</td>
<td>-14</td>
<td>Vertical stiffener. Main girder D</td>
</tr>
</tbody>
</table>

A better concurrence was found in FE-model 2, i.e. the finite element model with gaps between the deck and the steel top flanges.
In the same manner as for the cross girder at mid support and with a purpose to evaluate the bending moment of in one of the stiffeners, the stress along the top edge of the stiffener attached to main girder D was integrated.

As the vehicle was in a position far away from the girders, the impact from the weight of the vehicle (axial force) was assumed to be negligible. The bending moment due to the forced rotation was evaluated as:

- \( M = 20 \text{ KNm} \)

The bending moment corresponds to the following analytical stress in the stiffener top corner;

- \( \sigma = -39 \text{ MPa} \)

When comparing the bending moment in the stiffener at the intermediate cross girder (20 KNm) with the previously evaluated bending moment in the stiffener at mid support (35 KNm), it is seen that the intermediate cross girder seems to be less reluctant to bend when compared to the stiffer cross girder standing on bearings at mid support.

When rotations collected in the shell element finite model were imposed on a simple beam-element frame according to the figure below, the following bending moment was achieved in the stiffener attached to main girder D:

- \( M_{\text{girder D}} = 30 \text{ KNm} \)

Consequently a stiffer behaviour of the beam-element frame model than in the finite-element shell model.
4.2.2 Passage 018 on the cantilever of the deck
The vehicle is crossing the bridge in position at the cantilever of the deck, i.e. at a position not normally intended to host traffic.

Comparison with FE-analyses
As the force is transferred to the stiffener via the shear connectors, the magnitude of the stress is highly dependent on the position of the shear connectors’ relative to the stiffeners.

If the shear connectors are far away from the stiffener, the flexibility of the joints increases as the main girder flange deforms for forces perpendicular to the plane of the flanges and as the shear connectors elongate.

The joint thus becomes more non-rigid and the bending moment of restrain decreases. In context this means that the stiffener and cross girder will not experience a rotation of equal magnitude as the deck and a gap will open up between the deck and the top flange.

To study the effect of the above, the original FE-model was modified yet again, hence leaving us with another two, quite similar— but not identical— finite element models:

- **FE-model 3:**
  The shear connectors are modelled as beam elements and with shear connectors in alignment with the stiffeners

- **FE-model 4:**
  The shear connectors in alignment with the stiffeners were removed. Hence the two nearest shear connectors (relative to the position of the stiffener) were 150 mm before— respectively after the stiffeners
Global behaviour

The load distribution factor for main girder D was found to be around 1.0 meaning that main girder D is carrying almost the whole weight of the vehicle.

This fact is, in itself interesting as a bridge designer normally and for corresponding load positions would choose a load distribution factor equal to around 1.23 i.e. an overestimation of the load by 23 %

As the main girder D deflects, an almost negligible portion of the weight of the vehicle is transferred through the deck and the cross girder over to main girder C.

As in the previous section, the strains and corresponding stresses given by the two gauges (Ch. 4 & 8) again forms an almost perfect influence curve.

Also as in the previous section, an acceptable agreement between measurements and finite element analyses is found.
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**Stresses in vertical stiffeners at mid span**
The diagram below illustrates the strains (and the corresponding stresses) in Ch.5. As the truck at this particular time is passing by on the cantilever of the deck, the rotation of the deck is such that we encounter tensile stresses in the particular stiffener.

Upon unloading and as the concrete deck bounce back, an unexpected phenomenon occurs. The LDVT records a remaining gap/deformation between the top flange of the main girder and the deck, and this interlocking causes a force of compression in the vertical stiffener as the deck rotates back to its equilibrium position.

The most likely explanation for this event is some sort of internal “locking” within the system, possibly between the shear connectors and the concrete deck. A little later, the interlocking is released when another heavy vehicle pass by in between the two main girders.

The observed phenomenon is as an isolated event not important but is a reminder of the complexity of the junction between the steel and the concrete deck.

---

*Figure 71 Stiffener stresses at an intermediate cross girder. Passage 018*
The results from the FE-analysis are then being studied. One first observation when studying the deformation of the cross girder, is that possible effects due to a differential deflection between the main girders seems to be negligible. This is due to the fact that the torsional stiffness of the concrete deck is sufficient enough to make the out-of-plane deformations of the deck small. The cross girder is thus rotating around the longitudinal axis of the bridge almost like a rigid body.

*Figure 72 Movement between the deck and the top flange Passage 018*

*Figure 73 Deformation of the cross girder. Passage 018*
When studying the local deformation of the top part of the stiffener, it is seen that even if there, as in this case is a shear connector in direct alignment of the stiffener, the top flange of the main girder will still deform in such a way that a small gap or interstice is likely to appear.

Furthermore it is seen that the stress peak of tension in the top corner of the stiffener is very local. This is due to the fact that the stiffener top corner encounters the following:

- A stress peak due to the ordinary global bending of the stiffener
- A more- to its nature- local stress peak which in turn superposes the global bending stress. The local stress peak occur due to the fact that a tensile force is transmitted through the one shear connector on top of the stiffener and the force are transmitted into the stiffener in a single discrete point at the stiffener

At the Vårby Bridge, the shear connectors are transversely placed in particularly unfortunate positions as the positions coincide with the critical area, i.e. the top corner of the web stiffeners.

This is something that is not desirable at all and should be avoided when designing future bridges.

The tensile stress peak at the position of Ch.5 was evaluated as approximately 39 MPa and the force in the shear connector in alignment of the stiffener was found to
be 20 KN. The corresponding force in the two shear connectors 150 mm before-respectively after the stiffener was found to be 4 KN.

Figure 76 Stress at the position of Ch.5. A shear connector in alignment with the stiffener

The shear connector in direct alignment of the stiffener was then removed in the finite element model and the stress peak at the position of Ch. 5 was evaluated yet again.

Figure 77 Stress at the position of Ch.5. No shear connector is in alignment with the stiffener
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The effect of the removal of the shear connectors was found to be substantial. As the global bending moment in the stiffener is reduced, the gap between the steel and the concrete becomes larger. Furthermore, by omitting the one shear connector directly in alignment with the stiffener, the direct (and local) introduction of tensile force through the weld, into the stiffener is avoided. The stress peak after the removal was hence only found to be approximately 51% of the original stress. Compare the stress in figure 77 with the corresponding stress in figure 76.

The axial forces in the two shear connectors situated 150 mm respectively after the stiffener were approximately 8 KN.

No signs of high tensile stress in stiffeners at intermediate cross girders are found in the data achieved during the monitoring of the Vårby Bridge. But as only 2 intermediate cross girders were monitored (of a total of 56), it can't be out-ruled that loading on the cantilevers of the bridge decks could have been influencing the fatigue life of the welds.

A few conclusive observations are the following:

- A discrepancy between measured stresses and the finite element stresses are indeed found. It should be noted that the actual positions of the shear connectors' relative the stiffeners in the real structure are not known and are only guessed. To achieve a satisfying result, actual positions of shear connectors are important to know.

- The position of the test vehicle at the cantilever is also uncertain. This is due to the fact that the testing was conducted at night time and without the bridge being closed for traffic. The position of the vehicle cannot, in the same ways as when the vehicle positioned in between the main girders, be evaluated based upon global bending stresses in the main girder bottom flanges.

- Furthermore, while the deformability of the shear connectors are considered as well as the axial stiffness of the shear connectors, further flexibilities in the system is possible or even likely. For instance slips between the shear connectors’ shaft and plastic concrete, deformation beneath the head of the shear connectors. Such slips could be significant and further reduce the tensile stress peak.
4.3 Long term monitoring and evaluation of fatigue damage

To get a clearer understanding of the frequency and the magnitude of the strains and stresses affecting the welds daily; measurements were conducted for ordinary day-to-day traffic for a period of 104 h.

When studying the obtained stress spectrum; the following two conclusions can be made;

- The amount- and the magnitude of the fatigue relevant stresses at current areas of interest are sufficient enough to explain the cracks
- The fatigue relevant nominal loading stress cycles in the areas of interest are in principal all compressive

The obtained spectrum of varying stresses was reduced into sets of simple stress reversals and Rain-flow Counting-analyses (RFC) were performed. While performing the RFC, a threshold value $\Delta \sigma_{th}$ was adopted meaning that all stress ranges $< 10$ MPa were disregarded. The choice of the threshold or cut-off limit was based upon the observation that the total width of the welds is half of the corresponding width of the stiffeners. A stress range 10 MPa in the stiffener is equivalent to a stress range of approximately 20 MPa in the welds. A suggestion for a cut-off limit is the intersection of the fatigue strength curve ($m = 5$) at $10^8$ cycles (the fatigue limit). For the fatigue resistance class of interest (FAT 36) and for this particular case, that means a cut-off limit equal to 20 MPa.

![Stress peaks- Channel 5](image)

*Figure 78 Recorded stresses during the long term monitoring. Channel 5*
Based upon the result of the Rain-flow counting-analysis a damage summation according to the Palmgren-Miner rule was performed. The histogram for channel 5 & 14 according to the figures below and a complete set of histograms for every channel (1-23) are found in appendix A.

Using the Palmgren-Miner rule, it is possible to estimate the expected fatigue life (in years or in load cycles) of the welds, based upon the stress history achieved during the long term monitoring.

The rule states that the nominal fatigue life of a component or structure is exhausted and that failure (cracking) occurs when:

\[ \sum_{i=1}^{n} \frac{n_i}{N_i} \leq 1.0 \]  
(Eq. 16)

\( n_i \) is the number of applied load cycles of type \( i \), and \( N_i \) is the fatigue life expressed as number of cycles.

The fatigue life \( N_i \) is calculated as:

\[ N_i = \left( \frac{\Delta \sigma_i}{\Delta \sigma_c} \right)^m \cdot 2 \cdot 10^6 \]  
(Eq. 17)

Where \( \Delta \sigma_c \) is the fatigue strength for \( 2 \cdot 10^6 \) cycles, \( \Delta \sigma_i \) is the equivalent stress ranges evaluated in the performed Rain-Flow Counting analyses and \( m \) is the slope of the S-N curve.

In cases like this one, and in order to obtain a result as realistic as possible, it is reasonable to go beyond the frame of the design codes and make a few reasonable assumptions. Prior to the attempt to estimate the fatigue life for the welds between
vertical web stiffeners and top flanges at the Vårby Bridge, the following five assumptions were made:

1. The traffic intensity as well as the traffic distribution of the different types of vehicles passing the bridge at the particular period of 104 h was assumed to be representative from the day the bridge was taken into service until present day.

2. The chosen fatigue resistance is the FAT 36 according to EC 1993-1-9:2005. As the above fatigue class corresponds to a root crack in a fillet weld in a T-connection, loaded in the perpendicular direction to its length and presupposes a failure in the root of the weld and as this which is not the case at the Vårby Bridge where the cracks appear at weld toes, the chosen fatigue class FAT 36 could be questionable. Instead, the FAT 71 class according to the same code but referring to cracks at weld toes but in butt welds instead of fillet welds could be more appropriate.

Moreover, the above fatigue resistance class FAT 36 postulates a statistical safety margin of a 95 % survival probability which is a reasonable safety margin in the design phase where the aim is to ensure a sustainable structure. However, in a case like this, the safety margin is all too conservative.

In a review of the fatigue performance of fillet welds in cruciform joints, Maddox (2006) presented several S-N curves for a number of different tests. Among others the data used to derive the British Standard Class W and the data used to derive the EN fatigue category. For the purpose of this report; the fatigue data was reprocessed and a design curve of 50 % survival probability was considered as well as the 95 % standard assumption. For $2 \times 10^6$ load cycles a fatigue strength value of $\Delta \sigma_f = 70$ MPa was obtained. The S-N curve with a slope of 3.8 is indicated in figure 81.

![Cruciform Joint Test Data](image)

**Figure 81 Processed cruciform joint test data**
Kainuma et al performed a study on fatigue crack initiation points of load-carrying fillet welded cruciform joints where the influence of the weld size and the weld penetration were examined:

It was found that the fatigue strength of load-carrying joints increases with the weld size. According to Kainuma, the explanation is that, as the force in this type of joints is transmitted through the welds only, the stress concentration at the weld toes decreases as the weld size increases.

It was also found that the crack initiation point changed from the weld root to the weld toe at a weld size ratio of 1.2. According to the JSSC (Japanese Society of Steel Construction), the fatigue strength at 2 million cycles is 65 MPa in the main plate for a toe failure and 40 MPa in the weld throat for a root failure. This gives a critical weld size ratio of 1.15. Furthermore, it was found that the fatigue strength for both a root as well a toe failure increased with the weld penetration depth.

According to more tests upon fillet welded joints and performed by Kainuma et al. - this time under compressive cyclic stresses as in the case of the Vårby Bridge - fatigue cracks was found to initiate from the weld toes and in some cases also at the weld roots. The cracks were found to propagate to a length of 1 to 5 mm. Then the cracks ceased to propagate. It was also observed that the fatigue life under compressive cyclic loading was much longer than under a tensile cyclic loading [22].

3. All existing stress raisers were assumed to be included in the appropriate fatigue class
4. When transforming the stress ranges in the stiffeners, $\Delta \sigma_{\text{stiffener}}$, to equivalent stress ranges in the welds, $\Delta \sigma_{\text{weld}}$, the width of the weld were assumed to be wider, 6 mm, when compared to the design width of 5 mm. This is reasonable when taking into account that the real widths of fillet welds in practice almost always exceed the theoretical width prescribed at the design phase.

5. As the majority of the fatigue critical stress peaks in the area of interest are compressive and the welds of interest are fillet welds; the relationship between the amounts of force that is transmitted through the welds- or directly into the stiffener is uncertain. See the simplified figure below illustrating two extreme cases.

![Figure 84](image1) The welds are assumed to transfer 100% of the compressive force

![Figure 85](image2) The welds are assumed to transfer around 40% of the compressive force

In this report it is assumed that 100 % of all forces are transmitted by the welds.

Having appropriate fatigue strengths established as well as the stress ranges in the stiffeners, the equivalent stress ranges in the welds is left to be evaluated.

As the total thickness of the welds connecting the vertical stiffeners to the top flanges are only 50 % of the thickness of the stiffeners at mid span and 40 % at mid supports; the measured stresses in the stiffeners must be raised with a scalar factor $\alpha$;

$$
\alpha = \frac{\Delta \sigma_{\text{weld}}}{\Delta \sigma_{\text{stiffener}}} \quad (\text{Eq. 18})
$$

Assuming a linear stress distribution and taking the cope hole of a radius of 50 mm and the weld at the short end of the stiffeners into account $\alpha$ was estimated as;

$\alpha = 1.37$ for a web stiffener of thickness 20 mm  (intermediate cross girders)
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\( \alpha = 1.81 \) for a web stiffener of thickness 25 mm (mid-support cross girders)

The damage was then evaluated and extrapolated over a period of one year providing an estimation of the time of crack initiation. The complete result is presented in table 12.

**Table 12 Cumulative damage index evaluation. FAT 36, Pu 95 %**

<table>
<thead>
<tr>
<th></th>
<th>( \Delta \sigma )</th>
<th>( \Delta \sigma )</th>
<th>( n_i )</th>
<th>( n_i )</th>
<th>( \Sigma n_i/N )</th>
<th>( N )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder C</td>
<td>24.1</td>
<td>33.0</td>
<td>7268</td>
<td>612189</td>
<td>0.50</td>
<td>2.0</td>
</tr>
<tr>
<td>Girder D</td>
<td>25.1</td>
<td>34.4</td>
<td>11774</td>
<td>991733</td>
<td>1.03</td>
<td>1.0</td>
</tr>
<tr>
<td>Girder D</td>
<td>13.9</td>
<td>25.2</td>
<td>2183</td>
<td>183876</td>
<td>0.05</td>
<td>20.1</td>
</tr>
<tr>
<td>Girder D</td>
<td>25.9</td>
<td>46.9</td>
<td>9288</td>
<td>782335</td>
<td>2.10</td>
<td>0.5</td>
</tr>
</tbody>
</table>

\( \Delta \sigma_{\text{weld}} \) = calculated average stress range within the weld based upon the stiffener stress range

\( n_i \) = number of counts > 10 MPa for 104 hours

\( n_i \) = number of counts > 10 MPa for 1 year

\( \Sigma n_i/N \) = damage index for a period of 4 days, extrapolated over a period of 1 year

\( N \) = calculated length of time (years) from the erection of the bridge to the first appearance of a crack

If the more reasonable 50 % survival probability is used, the fatigue strength is accordingly raised with a factor equal to \( \frac{70}{36} = 1.94 \).

The damage index of the weld between the stiffener and top flange in position of Ch. 5 then drops to \( \frac{1.03}{1.94} = 0.14 \) for 1 year, implying a consumed fatigue life in approximately \( 1/0.14 = 7 \) years.

The complete result is presented in table 13 below.

**Table 13 Cumulative damage index evaluation. \( \Delta \sigma_c = 70 \text{ MPa} \) and Pu 50 %**

<table>
<thead>
<tr>
<th></th>
<th>( \Delta \sigma )</th>
<th>( \Delta \sigma )</th>
<th>( n_i )</th>
<th>( n_i )</th>
<th>( \Sigma n_i/N_i )</th>
<th>( N )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder C</td>
<td>24.1</td>
<td>33.0</td>
<td>7268</td>
<td>612189</td>
<td>0.07</td>
<td>14.7</td>
</tr>
<tr>
<td>Girder D</td>
<td>25.1</td>
<td>34.4</td>
<td>11774</td>
<td>991733</td>
<td>0.14</td>
<td>7.1</td>
</tr>
<tr>
<td>Girder D</td>
<td>13.9</td>
<td>25.2</td>
<td>2183</td>
<td>183876</td>
<td>0.01</td>
<td>148.0</td>
</tr>
<tr>
<td>Girder D</td>
<td>25.9</td>
<td>46.9</td>
<td>9288</td>
<td>782335</td>
<td>0.29</td>
<td>3.5</td>
</tr>
</tbody>
</table>

The fatigue life for Ch. 1 (7 years) and Ch. 5 (15 years) achieved when using the 50 % survival probability (\( \Delta \sigma_c = 70 \text{ MPa} \)) seems more reasonable when considering the fact that the cracks were discovered around 10 years after the bridge was taken into service.
4.4 Examination of a piece of weld containing a crack

In parallel to the monitoring of the bridge, a small steel sample, of a weld and adjacent base metal containing a crack was extracted from the bridge. The sample, which was examined at Luleå University of Technology [6], [7] and [8], was first sectioned into two parts denoted A and B and with the cut oriented perpendicular to the observed crack.

In part B and after a first visual inspection; only a minor surface crack was noticed. Hence part A was initially given the higher priority.

Examination of part A

Part A was first examined in an optical microscope. The most important observations are listed below;

- The local weld geometry is irregular and the profile of the weld is strongly curved which features contribute to stress concentration (for stress perpendicular to the crack plane)

- The initiation and propagation phases of the crack occurred exclusively in weld metal

- No flaws or other defects in the microstructure of the base metal of the stiffener, in the weld metal or in the HAZ (Heat Affected Zone) could be detected. The observations are all excluding faulty material and are instead indicating a geometrical and/or mechanical reason for the initiation of the crack.
Fractographic methods using SEM (Scanning Electron Microscope) were then used.

The remaining crack surfaces were found to be corroded. Near the origin of initiation, the corrosion was so severe that original surface texture of the crack was no longer clear.

However, the corrosion decreased with an increasing crack depth and at a depth of 2-3 mm along the crack plane, the crack surface was free from corrosion and striations and so called “beach marks” was seen. The presence of striations is a clear and unambiguous indication of crack growth by fatigue. Furthermore, the direction indicated by the striations was almost perpendicular to the global plane of the stiffener.

Figure 88 Illustration of the crack
The crack growth rate could be estimated based upon the distance between the striations and seemed to have been rapid. At a propagation rate as rapid as observed here, a fatigue crack would have penetrated the weld in less than 400,000 cycles.

The high propagation rate could possibly be explained by the occurrence of multiple initiations, meaning that (at least) two separate cracks had formed independently of each other and eventually merging into one crack. The phenomenon of multiple initiations is commonly associated with a large number of cycles.

Examination of part B
When broken up, an even larger crack compared to the first one was found. This observation confirmed the assumption of multiple initiations. As the two cracks were approaching each other, the propagation rate accelerated, thus the considerable propagation rate.

Conclusions
- The crack initiation and propagation occurred exclusively in the weld metal
- The cause and exact origin of the crack initiation is not fully known due to severely corroded crack surfaces. Crack initiation is however not due to faulty material, but is rather caused by geometrical and/or mechanical effects
- Striations and “beach marks” on the fracture surfaces are found, indicating unambiguously crack growth through fatigue
- Multiple initiations, i.e. at least two separate cracks have been formed independently of each other, and have eventually merged into one. The phenomenon of multiple initiation is commonly associated with a large number of cycles
4.5 Plausible cause of cracks and remaining fatigue life

The result of the investigation is unambiguously indicating material fatigue as being the cause to the cracks. All other, more or less possible causes are out ruled considering that:

- Striations and so called beach marks which both indicate crack growth through fatigue were found at the fracture surface of a crack in a small sample of a weld. The weld sample was collected from the bridge.
- The amount- and the magnitude of fatigue relevant stresses at areas of interest were found to be sufficient enough. In fact, the number of fatigue relevant stress peaks is numerous as in principle every sufficiently heavy vehicle contribute to the fatigue damage
- The fatigue life of welds at the Vårby Bridge, when using a 50 % survival probability for FAT 36 ($\Delta\sigma_C = 70$ MPa) was found to be 7 respectively 14 years. The result seems reasonable considering the fact that the cracks were discovered 10 years after the bridge was taken into service

The fatigue relevant stress at the origin of the cracks is believed to be the result of an imposed (or forced) rotation of rigid cross girders and web stiffeners. The rotation occurs when the slender concrete deck deflects as heavy vehicles pass by. This, to its nature secondary effect was probably overlooked at the design phase of the bridge.

As the majority of the stress peaks at points of interest were found to be compressive, and as fatigue cracks forms and grows in areas where the resultant stress is tensile; a presence of tensile residual stress at the origin of the cracks is assumed. Furthermore; the direction of the residual stress field should be perpendicular the crack planes. Research results supporting both assumptions are presented in the thesis.

Based upon the above assumptions, it is believed that- due to the residual stress at weld toes, superposing normally nominal compressive stress caused by live load, the mean stress of fatigue relevant stress cycles is raised to a level high up on the tension side. It is also believed that the shrinkage of the concrete - given certain circumstances -could contribute to raise the mean stress of fatigue stress cycles at points of interest.

The cracks are believed to constitute no immediate hazard for either the safety- or the sustainability of the bridge. In a worst case scenario and if the propagation continues; the stiffeners could in time become separated from the top flanges. If so - out of plane bending of the main girder web plates could occur, leading to what is popularly known as moustache cracks in the main girder web plates. This is not a desirable situation and must by all means be prevented.
As stated earlier, the crack growth process at the Vårby Bridge is believed to be the result of an effect due to the deflection of the concrete deck. This means that the cracking logically should be governed more by displacement than by real load. In contrast to a case where the cracking is governed by a real load; the stress intensity factor – a critical factor for the crack growth rate - decrease as the length of a crack increase. In context this implies a retarding crack growth process. It is therefore believed that the propagation of the observed cracks will slow down and eventually stop. The theory is illustrated by a small example below.

Consider two different modes of action of the external load. The load is assumed acting either under force controlled conditions or displacement controlled conditions. To conform to the situation of the observed cracks we assume a stress range sufficient for initiation of a fatigue crack in the weld at the outer free edge of the stiffener and propagation a short distance toward the main girder web. Naturally, upon crack propagation, the remaining weld area between the stiffener end and the main girder flange decreases. Under load controlled conditions, the stress in the remaining area increases accordingly, as the transmitted load is given and constant.

Consider a beam with cross section b x h and length a, which is also is the length of the crack.

![Figure 91](image)

The elastic modulus is E. The beam is clamped at one end and loaded with a transverse force P at the other end.

One half of the crack mouth opening is;

\[ \Delta = \frac{a^3}{3EI} \text{ and } l = \frac{bh^3}{12} \]  

(Eq. 19)

**Load controlled conditions**

The strain energy in one beam is;
The monitoring of the Vårby Bridge

\[
W = \frac{1}{2} \cdot P\Delta = \frac{1}{2} \cdot \frac{a^3}{3EI} \cdot P^2 \quad (\text{Eq. 20})
\]

And the total potential energy (in two beams) is;

\[
U = W - V = 2 \cdot \left(\frac{1}{2} \cdot P\Delta - P\Delta\right) = -P\Delta = -\frac{a^3 \cdot P^2}{3EI} \quad (\text{Eq. 21})
\]

The stress intensity factor can be obtained from;

\[
\frac{K^2}{E} = G = \frac{1}{b} \cdot \frac{dU}{da} = \frac{a^2}{bEI} \cdot P^2, \quad \text{or}
\]

\[
K = \frac{a}{\sqrt{b \cdot I}} \cdot P = 2 \cdot \sqrt{3} \cdot \frac{a}{bh\sqrt{h}} \cdot P \quad (\text{Eq. 23})
\]

The variation of \(K\) with crack length can be expressed by differentiating the above expression;

\[
\frac{dK}{da} = 2 \cdot \sqrt{3} \cdot \frac{1}{bh\sqrt{2}} \cdot \Delta > 0 \rightarrow Increasing \quad (\text{Eq. 24})
\]

It is seen that under load controlled conditions, the stress in the remaining area increases accordingly as the transmitted load is given and constant. This means that the stress intensity factor range of the crack increases and that the crack grow rate increases accordingly.

The instantaneous crack growth increment (crack growth per load cycle) becomes progressively greater than the adjoining previous increment. Crack growth is thus an accelerating process progressing until the maximum stress intensity factor attains the fracture toughness of the material. When that happens; failure ensues.

The number of load cycles from a given instant during the propagation process until failure ensues, can be taken as a measure of the remaining fatigue life of the component.

**Displacement controlled conditions**

Now turning the attention to the other mode of loading, it is noted that under controlled displacement conditions, the remaining area of the weld considered also decreases with crack propagation.

However, unlike controlled loading, the stress in the remaining area, in particular in the material ahead of the crack tip, does not increase under controlled displacement...
conditions. Instead the stress will in fact decrease, as the controlled deformation decreases with increasing crack length.

The total potential energy (in the same two beams) is:

$$U = W = 2 \cdot \frac{1}{2} P \Delta = \frac{3EI}{a^3} \cdot \Delta^2$$  \hspace{1cm} (Eq. 25)

Accordingly, the stress intensity factor is obtained from:

$$K^2 = G = -\frac{1}{b} \cdot \frac{dU}{da} = 9 \cdot \frac{EI}{b \cdot a^3} \cdot \Delta^2, \quad \text{or}$$

$$K = 3 \cdot \frac{E}{a^2} \sqrt{\frac{1}{b} \cdot \Delta} = \frac{\sqrt{3}}{2} \cdot \frac{E \cdot h}{a^2} \cdot \sqrt{h} \cdot \Delta$$  \hspace{1cm} (Eq. 26)

The variation of $K$ with crack length is given as:

$$\frac{dK}{da} = -\sqrt{3} \cdot \frac{Eh}{a^3} \cdot \sqrt{2} \cdot \Delta < 0 \rightarrow \text{Decreasing}$$  \hspace{1cm} (Eq. 28)

This means that the stress intensity factor range decreases and the crack growth rate decreases accordingly with increasing crack length.

It should be noted that the above assumption of a decrescent crack growth have not been tested by for example fracture mechanic methods. Using such methods, the remaining fatigue life, $N_f$, can be evaluated, expressed as number of cycles from crack initiation until total fracture.

The analysis would then be done according to the following procedure:

The stress intensity factor range $\Delta K$ is described as:

$$\Delta K = Y \cdot \Delta \sigma \cdot \sqrt{\pi} \cdot a$$  \hspace{1cm} (Eq. 29)

$a$ \hspace{1cm} crack size

$\Delta \sigma$ \hspace{1cm} cyclic stress range

$Y$ \hspace{1cm} The product of various multipliers which account for the geometry of the cracked body

Using the well-known Paris law; the applied stress intensity factor range can be related to the crack growth as follows;
The monitoring of the Vårby Bridge

\[
\frac{da}{dN} = C \cdot \Delta K^m \quad (Eq. 30)
\]

- \(C\): constant of the Paris law
- \(m\): exponent of the Paris law

By integrating the Paris law over the crack size, the fatigue life of the detail can be evaluated.

\[
N_f = \int_{a_0}^{a_c} \frac{da}{\frac{C}{\Delta K^m}} \quad (Eq. 31)
\]

- \(N_f\): The number of cycles needed from crack initiation to failure
- \(a_0\): initial crack size
- \(a_c\): final or critical crack size

One large limitation is that their application for “distortion-induced” fatigue cracking due to secondary stresses are rather complex [21].

Another large limitation is modeling issues due to the fact that we are dealing with compressive stresses which are superposed by residual stresses of unknown value and distribution. Therefore, the possibility of an evaluation of remaining fatigue life for the welds at the Vårby Bridge was taken under consideration but was eventually considered as less suitable for this particular case.

During the installation of the monitoring equipment and with the purpose to ensure a future control of the propagation phase at the Vårby Bridge, one crack (the largest one) was selected and marked to allow its further growth to be followed. It is strongly recommended that the bridge owner continues to inspect the crack, tentatively once a year.
5 Lab Testing of a Composite Bridge

5.1 Introduction

In the summer of 2011, lab testings were performed in cooperation with the RFCS project “ELEM” (Element Bridges). The tests were carried out at Luleå University of Technology (LTU).

A composite bridge of a length 7.2 m and width 3.5 m was subjected to different kinds of testing. For the purpose of this thesis; a large number of strain gauges were installed and a few static tests were performed on two different types of cross girders, type 1 and type 2. Both types commonly used in composite bridges in Sweden.

The testing was finalized with a cyclic test performed upon one of the cross girders.
5.2 Test structure

Main steel structure

Table 14 Steel plates of the main girders

<table>
<thead>
<tr>
<th>Position</th>
<th>Part</th>
<th>Dimensions</th>
<th>Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Top flange</td>
<td>Plate 20-300</td>
<td>S355J2</td>
</tr>
<tr>
<td>2</td>
<td>Web plate</td>
<td>Plate 12-760</td>
<td>S355J2</td>
</tr>
<tr>
<td>3</td>
<td>Bottom flange</td>
<td>Plate 20-300</td>
<td>S355J2</td>
</tr>
</tbody>
</table>

Cross girder type 1 with web stiffeners

Cross girder type 1 consisted of a single hot-rolled girder, UPE 300, attached to the vertical stiffeners on the main girders by bolts. The thickness of the vertical stiffeners was 20 mm and the width was 135 mm. The stiffeners were welded to the main girder plates by fillet weld of a width of 5 mm.
Cross girder type 1 was constructed in so that it could emulate both an intermediate cross girder as a cross girder at supports. For that purpose, one additional web stiffener was welded on the outside of the web.

**Cross girder type 2 with web stiffeners**

The 2nd type of cross girder used at the testing's was a so called “K-truss”. A common used type of intermediate cross girder all over the world. The type are to prefer where the height of the main girders are sufficiently high. This was not the case here as the height of the main girders were only 800 mm. The top- and bottom chord as well as the two diagonals were designed as RHS-profiles according to the table below:

<table>
<thead>
<tr>
<th>Part</th>
<th>Profile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top chord</td>
<td>RHS 70x70x4</td>
</tr>
<tr>
<td>Diagonals</td>
<td>RHS 50x50x5</td>
</tr>
<tr>
<td>Bottom chord</td>
<td>RHS 70x70x4</td>
</tr>
</tbody>
</table>

![Figure 95 Cross girder type 2](image)

![Figure 96 Modified cross girder type 2. Top chord removed](image)
One recently used option to reduce the bracing at cross girder connections, is to remove the top chord after the deck has been casted. Therefore, and with a purpose to evaluate the effect of such a manoeuvre, the top chord of the truss was removed during one of the static tests.

The picture to the left show the Bridge over the lake of Ulvsunda, a double track bridge for commuter trains in Stockholm Sweden where the top chord of all intermediate cross girders was removed after the casting of the deck was performed.

The measure was found to significantly reduce the resistance against the deck deformation for the live loads. To further reduce the bracing, the width of the vertical stiffeners was reduced to a minimum. At the very connection of stiffener to top flanges, the width of the stiffener increased with a smooth transition to a width 230 mm, i.e. a width almost the same as the width of the top flange.

Most importantly, the welds between the stiffeners and the top flanges were prescribed as butt welds.

**Assembled steel structure**

The picture below illustrates the assembled steel structure for the testing bridge.
Concrete slabs
The deck of the bridge consisted of four prefabricated concrete elements with “dry joints” in the sense that the joint between each element is not cast together. Neither is reinforcement bars crossing the joints. Vertical forces are instead transferred from one element to the next by a system of “male-female” overlapping concrete keys. The connection between the slabs and the steel main girders beneath are achieved through channels inside each element which are injected with concrete when all elements are in their final position. One such channel is seen in figure 100.

Figure 99 Plan view of a prefabricated concrete slab

Figure 100 Channel in the concrete slab
Transverse reinforcement bars inside the elements are passing through the channels between the shear stud connectors hence providing the necessary interaction between the girders and the concrete slabs after the channels have been casted.

**Shear connectors**
The shear connectors used to achieve composite action between the steel main girders and the prefabricated concrete deck slabs were the standard $\Phi 22 \times 175$, which is a commonly used type of headed shear connector in Swedish composite bridges.

The lateral distance between the two shear connector’s rows on the top flange of the main girder was 150 mm. In the longitudinal direction and in order to match the arrangement of the reinforcement in the concrete elements, the shear connectors was placed in bundles of 22 with an internal distance of 1800 mm between the bundles.

The distance between each shear connector within the bundles was 150 mm; hence the number of connectors was $2 \cdot \frac{1}{0.15} = 13.3$ per meter flange.

**Sectional data and capacities**
The main girders were designed to withstand a bending moment $M = 2.92$ MNm, not including the bending moment of dead loads. The shear force capacity of the main girder web plates was 1.78 MN.

As the number of shear connectors on each top flange were 13.3 per meter and the shear force capacity for one stud is 0.109 MN (= Shear capacity for one connector according to EC 1994-2:2005), the allowable shear force in one main girder web with the respect to the shear flow between the deck and the steel top flange was a bit lower, 1.19 MN.
5.3 Measuring equipment

A total number of $34 + 12 = 46$ strain gauges were installed. The approximate position of each and one of the gauges can be seen in figure 101 and 102.

**Positions of the strain gauges at Cross girder type 1**

![Figure 101 Strain gauges at cross girder type 1](image)

The three stiffeners and the UPE profile forming cross girder type 1 were equipped with a total number of twenty strain gauges, denoted number 15 to number 34.

As the loading during the testing always would be in a position in-between the two main girders; the rotation of the deck was such that tensile normal forces was expected in the shear connectors outside of the main girder web plate.

In particular the shear connector directly in alignment of the additional stiffener located on the outside of the main girder web plate. For that purpose, three shear connectors denoted A, B and C were equipped with four strain gauges each according to the figure above.

**Positions of the strain gauges at Cross girder type 2**

![Figure 102 Strain gauges at cross girder type 2](image)
Measurement of rotations

In order to evaluate the transverse rotation angle during the tests, LDVT gages were used to register the vertical displacement $\Delta$, according to the figure to the below;

\[ \theta = \frac{(\Delta_2 - \Delta_1)}{500} \]  \hspace{1cm} (Eq. 32)
5.4 Finite element model

The structure for the finite element model used to recapture the testing's was in principle the same as for corresponding models used for the Vårby Bridge assessment.

Yet again, the top flanges of the main girders and the concrete deck was tied together by so called rigid links except in areas where tensile forces were expected to be transferred to the steel.

In those areas, the rigid links were substituted by beam elements corresponding to the shear connectors on the steel top flanges.

![Figure 104 FE-modell. Concrete omitted](image-url)
5.5 Tests results and comparison with finite element analyses

5.5.1 Setup 1 – Static test

The first static test was conducted on cross girder type 1, i.e. the cross girder consisting of a UPE-profile.

The load was in the lengthwise position placed directly over the cross girder, see figure 105 below.

In the transverse direction, the load was placed centred between the two main girders, see figure 106.
The bridge was loaded progressively in ranges of 100 KN. From a minimum load $P_{\text{min}} = 100$ KN to a maximum load $P_{\text{max}} = 500$ KN. In between each subsequent loading, the bridge was relieved as the load was lowered to 10 KN.

As the response of the steel structure (i.e. the cross girder and the web stiffeners) is similar (linear) for each one of the five loading steps (100-500 KN), only the results (strains and displacements) for the load $P = 300$ KN are presented here.

The following was recorded during the test and chosen data, listed below is reproduced in the following sections;

- Strains in a number of strain gauges in different positions at the vertical stiffeners and the cross girder. In the following sections; the strains [\(\mu\text{m/m}\)] are translated to and presented as stress [MPa]

- Strains in a total of twelve gauges distributed evenly at three shear connectors in position according to section 5.3 and figure 97 but similarly as above, the strains [\(\mu\text{m/m}\)] are translated to and presented as stresses [MPa]

- The transverse rotations [\(\%\)] of the concrete deck directly above the center of two main girders at the position of cross girder type 1

Moreover, the corresponding results from the performed FE-analysis are presented simultaneously with the result obtained during the measurement.
Stiffener stresses and comparison with FE-analysis

The following figures illustrate stresses at positions of strain gauge 15-18, 21-24 and 27-30. Moreover, in figure 110, the measured rotations of the two stiffeners are seen.

One direct observation is that the rotation of the deck above the girder equipped only with a single web stiffener is considerably higher than the corresponding rotation above the girder with two web stiffeners.

The relative difference of rotation is about 20% higher above main girder A (= 0.40/0.33 = 1.20).

The observation is not surprising as the cross section with two stiffeners is stiffer in comparison to the cross section with only one stiffener. In fact, the difference in rotation corresponds well with the corresponding relative difference in stiffness.

If a strip of a width of 500 mm of the main girder web for simplicity are assumed to participate together with the stiffeners in an effective cross sectional area and at
sections equal to the upper edges of the stiffeners; the following moment of inertia are achieved.

- Main girder A: \( I = 1.72 \cdot 10^7 \text{ mm}^4 \)
- Main girder B: \( I = 2.07 \cdot 10^7 \text{ mm}^4 \)

It is seen that the relative difference is approximately 20 % higher \((= \frac{2.07}{1.72} = 1.20)\)

As the bending moment above both main girders should be of equal magnitude and as the relative difference of stiffness of the two stiffener cross sections is 20 %, the relative difference in stress should also be 20 %.

The above assumption is almost fulfilled as the relative difference of stress at the position of Ch. 15 and Ch. 21 is \(\frac{46}{39} = 1.18\), i.e. 18 % higher stress at the position of Ch. 15, the strain gauge located at the cross section with only one vertical stiffener.

When evaluating the corresponding stress peak in the FE-model, a more concurrent result between measured stress and FE-stress were found then was the case in Chapter 4.

![Deformation of the cross girder. Setup 1](image)

\textbf{Figure 111} Deformation of the cross girder. Setup 1
The numeric analysis predicts stresses at critical areas slightly higher than the corresponding measured stresses.

The relative difference in stress between the stress at position of Ch. 15 and the stress at position of Ch. 21 was found to be approximately as the predicted 20% as \( \frac{55.7}{46.7} = 1.21 \) however the rotation of the deck in the finite element model deviated as the same rotation (0.34 %) was noted above each one of the two main girders.
Table 16 Comparison between measured stress and rotation and corresponding from FE-model

<table>
<thead>
<tr>
<th>Channel/Girder</th>
<th>$\theta_{\text{measure}}$</th>
<th>$\theta_{\text{FE}}$</th>
<th>$\sigma_{\text{measure}}$</th>
<th>$\sigma_{\text{FE}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>-</td>
<td>-</td>
<td>-46</td>
<td>-56</td>
</tr>
<tr>
<td>21</td>
<td>-</td>
<td>-</td>
<td>-39</td>
<td>-47</td>
</tr>
<tr>
<td>27</td>
<td>-</td>
<td>-</td>
<td>0</td>
<td>3</td>
</tr>
<tr>
<td>30</td>
<td>-</td>
<td>-</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>Girder A</td>
<td>0.00039</td>
<td>0,00034</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Girder B</td>
<td>0.00032</td>
<td>0,00034</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Shear connectors

As one of the main girders was equipped with one additional stiffener on the outside of the main girder web plate and for the particular load setups, it was assumed that tensile forces would be transferred into the stiffener by way of the shear connectors.

Each one of the three chosen shear connectors were equipped with four strain gages and the configuration made it possible to register both a possible transversal- as well as longitudinal strain gradient in each shear connector. Naturally, the average strain in each shear connector could also be evaluated.

In table 17, measured strains are presented for shear connector B, i.e. the one shear connector situated directly on top of the additional web stiffener on the outside of the main girder web plate.

The strains according to table 17 are used to calculate the axial force and the bending moment in the shear connectors using the relations according to figure 115.
The following strains $\varepsilon_{\text{average}}$ were registered:

**Table 17 Measured strain in shear connector A, B & C.**

<table>
<thead>
<tr>
<th></th>
<th>$\varepsilon_{\text{measure}}$</th>
<th>$\varepsilon_{\text{average}}$</th>
<th>$\sigma_{\text{axial}}$</th>
<th>N</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>293</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>B2</td>
<td>154</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>B3</td>
<td>35</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>B4</td>
<td>187</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(B1+B3)/2</td>
<td>164</td>
<td>34</td>
<td>13</td>
<td></td>
</tr>
<tr>
<td>(B2+B4)/2</td>
<td>171</td>
<td>36</td>
<td>14</td>
<td></td>
</tr>
</tbody>
</table>

The strain corresponding to the axial force, $\varepsilon_{\text{average}}$, in the shear connector shaft can thus be calculated as:

$$\varepsilon_{\text{average}} = \frac{\varepsilon_{B1} + \varepsilon_{B3}}{2} = \frac{\varepsilon_{B2} + \varepsilon_{B4}}{2} \quad \text{(Eq. 33)}$$

With $\varepsilon_{\text{average}}$ known, the axial force $N$ is given as:

$$N = \varepsilon_{\text{average}} \cdot E \cdot A = \sigma_{\text{axial}} \cdot A \quad \text{(Eq. 34)}$$

$A$ is the Area of the shear connector shaft $= \pi \cdot 11^2 \text{ mm}^2$.

The normal force in shear connector B was found to be approximately 13 KN. The force corresponds well with the analytic evaluation of the same and based upon the registered rotation of the deck.

$$N = \frac{\theta \cdot d \cdot EA}{L} = \frac{0.00032 \cdot 75 \cdot 210 \cdot \pi \cdot 11^2}{155} = 12.4 \text{ kN} \quad \text{(Eq. 35)}$$
\( \theta \) is the rotation angle, \( d \) is the distance between the centre of the main girder and the centre of the shear connector and \( L \) is the length of the shear connector shaft.

\[ \varepsilon = \delta \frac{d}{L} \]

Mainly due to the fact that the top flange of a thickness 20 mm is not rigid for a force perpendicular its own plane, the axial force in shear connector A and C are smaller, approximately \( \frac{1}{4} \) of the corresponding force found in shear connector B.

The transverse bending of the shear connector was also found to be significant. As the bending strain and stress at a section 30 mm above the main girder top flange is:

\[ \varepsilon_{\text{bending}} = \pm (\varepsilon_{\text{Measure}} - \varepsilon_{\text{average}}) \Rightarrow \sigma_{\text{bending}} = \pm 27 \text{ MPa} \quad (\text{Eq. 36}) \]

The aggregate stress in the shear connector and at that section can thus be estimated as:

\[ \sigma = \sigma_{\text{axial}} + \sigma_{\text{bending}} = 62 \text{ MPa} \quad (\text{Eq. 37}) \]

Shear connectors are not designed to transfer axial forces and to the best knowledge of the author of this thesis, the fatigue strength of shear connectors welded to a flange and subjected to axial forces and bending are not known. But it is not unlikely that a cyclic stress of such a magnitude could cause a fatigue crack.

In this context it should also be noted that the magnitude of the rotation (0.32 \%o) of the deck is not remarkably high and that the transverse distance between the centre of the main girder and the shear connector in question is only 75 mm, a distance that must be considered as narrow.
The problem with cyclic forces in the shear connectors being transferred to the steel main girders beneath have been studied for example by Japanese researchers. Sakurai et.al [12] performed both analytical and experimental studies.

Figure 118 Axial force in shear connectors measured at an actual bridge. Taken from [12]

Sakurai made the conclusion that it is desirable to avoid the presence of shear connectors in positions just above cross girder connections.

Another conclusion made by Sakurai is that it is necessary to provide a certain extent of stiffness to the top flange, with respect to bending deformation of the top flanges of main girders.

In Switzerland, testings have been performed upon a miniature composite bridge. The purpose of the testing was to obtain an experimental database to be used to calibrate numerical models.

Two different types of stiffeners were tested;

1. A single stiffener
2. Stiffeners composed by two separate H-profiles cut in half.

The 2nd case is for obvious reasons much more rigid for the transverse bending of the concrete slabs.

A cross section of the testing bridge at one cross girder is seen in figure 119.
It was found that the biasing of the shear connectors was much greater for the stiffeners of the 2\textsuperscript{nd} type. In fact, remarkably large forces were registered in the shear connectors. Compare with the table below:

<table>
<thead>
<tr>
<th>Stud</th>
<th>Applied force</th>
<th>N</th>
<th>M</th>
</tr>
</thead>
<tbody>
<tr>
<td>g2</td>
<td>169.7</td>
<td>119.2</td>
<td>143.3</td>
</tr>
<tr>
<td>g3</td>
<td>122.7</td>
<td>117.4</td>
<td>157</td>
</tr>
<tr>
<td>g4</td>
<td>170.4</td>
<td>112.1</td>
<td>164</td>
</tr>
<tr>
<td>g10</td>
<td>162.9</td>
<td>73.4</td>
<td>268.1</td>
</tr>
</tbody>
</table>

Studying the result during the testing performed at LTU and the result achieved by Sakurai et.al and the Suisse testing of a miniature composite bridge, it is clear that attention should be made to both longitudinal and transversal positions of shear connectors relative vertical web stiffeners.

The introduction of tensile force, transferred by shear connectors into the steel main girder, can be avoided. For example by the following measures:

A. Place the shear connectors, between the concrete deck and the main girders, in close transversal spacing in order to avoid an introduction of a large bending moment into the main girder at a stiffener.
The above action (A) was also suggested by Greiner et.al [2]

B. Avoid shear connectors near- or even in alignment with vertical web stiffeners. This option will reduce the global bending moment in the stiffener. Furthermore, a direct introduction of tensile force perpendicular the welds is avoided.

The above action (B) was also suggested by both Greiner and Sakurai et.al [12]

C. Reduce deck rotations by making the deck sufficiently thick enough. The action was also suggested by Greiner.

The rotations could also effectively be reduced by for example integrating cross girders at mid-supports with the concrete deck
5.5.2 Setup 2 – Static test

The 2nd static test was conducted on cross girder type 2 i.e. the cross girder consisting of RHS-profiles and with the upper chord intact.

The load was rearranged to a longitudinal position in alignment with the cross girder in question. The transversal position of the load was identical as for setup 1.

![Figure 120 Longitudinal position of the load. Setup 2](image1)

![Figure 121 Transversal position of the load. Setup 2](image2)
Stiffener stresses

**Figure 122** Measured stresses. Setup 2 and Ch. 1-4

**Figure 123** Measured stresses. Setup 2 and Ch. 9-12

**Figure 124** Rotation of the deck. Setup 2

**Figure 125** Measured stresses in the top chord. Setup 2 and Ch. 5-8

**Figure 126** Axial force in top chord. Setup 2
Stiffener stresses and rotations

A first distinct observation when studying figure 122-126 is that the stiffener rotations for cross girder type 2 is significantly larger in comparison to corresponding rotations for cross girder type 1. It can thus be concluded, that the stiffness of connected bracing elements, i.e. vertical stiffeners and adjoining cross girders, are influencing the very magnitude of the rotation. At least it is so for this particular case.

When studying the rotation of the two stiffeners within the cross girder frame, it is clear that the rotation above main girder A is significantly larger than the rotation above main girder B. The rotation is in fact almost 1,5 times greater as \( \frac{\theta_A}{\theta_B} = \frac{0.67}{0.45} = 1.49 \). This should reasonably be deduced to the fact that the stiffener attached to main girder A is not as wide as the stiffener attached to main girder B and hence is less resilient to bend.

The relative difference in stiffness between the two stiffeners can roughly be approximated as; \( \left( \frac{b_{\text{stiffener B}}}{b_{\text{stiffener A}}} = \frac{135}{100} \right)^3 = 2.46 \)

A discrepancy is found between rotations obtained from measurement and corresponding obtained in finite element model as the rotations achieved in the finite element model indicate rotations of almost equal value above each main girder. Moreover, the numerical magnitudes of rotations achieved in the finite element are much smaller than the measured ones. Whether the observed discrepancy is indicating some kind of internal restrain within the real bridge system, or simply an inadequate finite element modelling is not to 100 % clear. It should in this context be noted that the finite element results, both rotations and stresses, achieved in section 5.5.1 were considered as reasonable when compared to corresponding measured data.
When turning attention to measured stiffener stresses, it is seen in figure 122, that the intense stress peak, usually found in the outmost top corner of the stiffener, is now reduced to a minimum as the stress in position of Ch. 1 is only -6 MPa. As predicted, the stress concentration is instead found at the lower quadrant of the radius (or cut-out) where the stress peak in position of Ch. 4 is intense; almost -81 MPa. In a real construction this could possibly be accepted as the fatigue class is much more lenient.

There is a high stress peak as well in the position of Ch.3 (see figure 122). The position of that channel is in principal the extension of the outmost stiffener edge towards the top flange. The stress peak at the position of Ch.3 was found to be -37 MPa, to be compared to the stress peak at position of Ch.9 (see figure 122), which is -41 MPa. It should be remembered that the stiffener attached to main girder A endures almost a 1,5 times larger rotation. Furthermore, a lot is won merely by the fact that the stress peak is transferred from the critical area- the turn-around weld at the stiffeners free edge- towards the web of the main girder. Intuitively, that should be of benefits for the weld why a conclusion is that the design option of the one stiffener attached to main girder A seems promising.

The corresponding stress peaks achieved in the finite element model are seen in figure 128 & 129 and in table 19.
Table 19: Comparison between measured stresses and FE-model stresses. Ch. 1,2,3,4 and 9

<table>
<thead>
<tr>
<th>Channel/Girder</th>
<th>$\theta_{\text{Measure}}$</th>
<th>$\theta_{\text{FE}}$</th>
<th>$\sigma_{\text{Measure}}$</th>
<th>$\sigma_{\text{FE}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-</td>
<td>-</td>
<td>-6</td>
<td>-18</td>
</tr>
<tr>
<td>2</td>
<td>-</td>
<td>-</td>
<td>-27</td>
<td>-22</td>
</tr>
<tr>
<td>3</td>
<td>-</td>
<td>-</td>
<td>-37</td>
<td>-24</td>
</tr>
<tr>
<td>4</td>
<td>-</td>
<td>-</td>
<td>-81</td>
<td>-51</td>
</tr>
<tr>
<td>9</td>
<td>-</td>
<td>-</td>
<td>-41</td>
<td>-50</td>
</tr>
<tr>
<td>Girder A</td>
<td>0.00066</td>
<td>0.00037</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Girder B</td>
<td>0.00045</td>
<td>0.00036</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
5.5.3 Setup 3 – Static test
The 3rd static test was in principal identical to 2nd test, but with one substantial difference, the upper strut (top chord) of the truss was removed.

Figure 130 Longitudinal position of the load. Setup 3

Figure 131 Transversal position of the load. Setup 3
When compared to the previous section, 5.5.2, it is seen that the rotation of the stiffener attached to main girder A has increased but not nearly as much as was expected.

The rotation with the top chord intact was found to be 0.66 ‰, to be compared to 0.67 ‰ with the top chord removed. It is thus evident that the removal of the top chord, for this particular case, is of utterly minor influence.

As this, by all reasoning shouldn’t be a general truth, it is concluded that the chosen profiles for the RHS-profiles probably were all too slender and that the truss cross girder was not behaving as intended as the cross girder frame as a whole was too weak.

This was also observed during the testings’ and in finite element analyses as significant out-of-plane bending of the structure was observed, see figure 135.
Some minor effects of the removal of the strut are however seen and are listed below;

- The peak stress at position of Ch. 3 has increased. This is probably due to a larger rotation
- The peak stress at position of Ch. 9 has increased. This is also probably due to a larger rotation
- The peak stress at position of Ch. 4 has decreased. This is probably due to the removal of the bracing top chord

As in the section 5.5.2, the same discrepancy is found between rotations obtained from measurement and the corresponding obtained in the finite element model.

The finite element model stress for Ch. 1, 4 & 9 are seen on the following page
Figure 136 FE stresses in position of Ch. 1 & 4
Figure 137 FE stress in position of Ch. 9

<table>
<thead>
<tr>
<th>Table 20</th>
<th>Comparison between measured stresses and FE-model stresses. Ch. 1, 3, 4 &amp; 9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Channel/Girder</td>
<td>$\theta_{\text{Measured}}$</td>
</tr>
<tr>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>-</td>
</tr>
<tr>
<td>9</td>
<td>-</td>
</tr>
<tr>
<td>Girder A</td>
<td>0.00067</td>
</tr>
<tr>
<td>Girder B</td>
<td>0.00053</td>
</tr>
</tbody>
</table>
5.5.4 Setup 4 – Cyclic test
The static tests were complemented with a cyclic test performed on cross girder type 1, in an attempt to force a fatigue crack to appear. The cross girder in question (cross girder type 1) was subjected to a cyclic load ranging between a minimum load $P_{\text{min}} = 100$ KN and a maximum load $P_{\text{max}} = 450$ KN.

Between the 26th of August to the 21st of September 2011, a total number of $1,144,733$ cycles were executed, distributed on 16 different blocks (occasions). The frequency of the load was set to 5 Hz i.e. approximately 18,000 load cycles every hour.

The main focus was put on the strain gauge denoted Ch. 15 situated at the single vertical stiffeners attached to the main girder A, and on strain gauge B1 situated on the shear connector on top of the outermost stiffener, see figure 138 above. The stress range $\Delta \sigma_{\text{stiffener}}$ in the stiffener plate section was in average 62.8 MPa at the position of channel 15. The stiffener stress range should correspond to a stress range $\Delta \sigma_{\text{weld}} = 98.6$ MPa, perpendicular the fillet weld located between the stiffener and the main girder top flange.

The maximum stress range in the shear connector at the position of gauge B1, including the contribution of axial force as well as bending was 94 MPa, see figure 140.
A cumulative damage index evaluation for the weld between the vertical stiffener and top flange in position of channel 15 was then performed. Prior to the evaluation, the following assumptions were made:

1. Stress concentrations from the weld itself was assumed to be included in the fatigue class (FAT 36 according to the EC 1993-1-9:2005)
2. A tensile residual stress was assumed to exist within the weld
3. The fatigue strength $\Delta\sigma_c$ was based upon the last cycle as described by the following expression below:

$$\Delta\sigma_c = \Delta\sigma_{c,36} \cdot \left(\frac{2 \cdot 10^6}{n_t}\right)^{1/3}$$  \hspace{1cm} \text{Eq. 38}

<table>
<thead>
<tr>
<th>Block</th>
<th>$n$</th>
<th>$\Delta\sigma_{c,10}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Block 1</td>
<td>51 798</td>
<td>43.4</td>
</tr>
<tr>
<td>Block 2</td>
<td>168 606</td>
<td>43.4</td>
</tr>
<tr>
<td>Block 3</td>
<td>276 714</td>
<td>43.4</td>
</tr>
<tr>
<td>Block 4</td>
<td>360 434</td>
<td>43.4</td>
</tr>
<tr>
<td>Block 5</td>
<td>363 350</td>
<td>43.4</td>
</tr>
<tr>
<td>Block 6</td>
<td>373 350</td>
<td>43.4</td>
</tr>
<tr>
<td>Block 7</td>
<td>458 426</td>
<td>43.4</td>
</tr>
<tr>
<td>Block 8</td>
<td>514 528</td>
<td>43.4</td>
</tr>
<tr>
<td>Block 9</td>
<td>576 130</td>
<td>43.4</td>
</tr>
<tr>
<td>Block 10</td>
<td>826 130</td>
<td>43.4</td>
</tr>
<tr>
<td>Block 11</td>
<td>864 130</td>
<td>43.4</td>
</tr>
<tr>
<td>Block 12</td>
<td>926 130</td>
<td>43.4</td>
</tr>
</tbody>
</table>
The result of the damage summation is seen in the figure and the table below:

### Figure 141 Cumulative damage for the weld between the top flange and the vertical stiffener

### Table 22 Cumulative damage index evaluation for the weld between the stiffener and top flange

<table>
<thead>
<tr>
<th>Block</th>
<th>$\Delta \sigma_{\text{stiffner}}$</th>
<th>$\Delta \sigma_{\text{wld}}$</th>
<th>$n$</th>
<th>$\Delta \sigma_{16}$</th>
<th>$N$</th>
<th>$D = \Sigma (n/N)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bl. 1</td>
<td>61</td>
<td>96</td>
<td>51 798</td>
<td>43.4</td>
<td>187 046</td>
<td>0.28</td>
</tr>
<tr>
<td>Bl. 2</td>
<td>61</td>
<td>96</td>
<td>116 808</td>
<td>43.4</td>
<td>187 046</td>
<td>0.62</td>
</tr>
<tr>
<td>Bl. 3</td>
<td>61</td>
<td>96</td>
<td>108 108</td>
<td>43.4</td>
<td>187 046</td>
<td>0.58</td>
</tr>
<tr>
<td>Bl. 4</td>
<td>60</td>
<td>94</td>
<td>83 720</td>
<td>43.4</td>
<td>197 064</td>
<td>0.42</td>
</tr>
<tr>
<td>Bl. 5</td>
<td>60</td>
<td>94</td>
<td>2 916</td>
<td>43.4</td>
<td>197 064</td>
<td>0.01</td>
</tr>
<tr>
<td>Bl. 6</td>
<td>60</td>
<td>94</td>
<td>10 000</td>
<td>43.4</td>
<td>197 064</td>
<td>0.05</td>
</tr>
<tr>
<td>Bl. 7</td>
<td>60</td>
<td>94</td>
<td>58 076</td>
<td>43.4</td>
<td>197 064</td>
<td>0.43</td>
</tr>
<tr>
<td>Bl. 8</td>
<td>63</td>
<td>99</td>
<td>56 102</td>
<td>43.4</td>
<td>168 958</td>
<td>0.33</td>
</tr>
<tr>
<td>Bl. 9</td>
<td>63</td>
<td>99</td>
<td>61 602</td>
<td>43.4</td>
<td>168 958</td>
<td>0.36</td>
</tr>
<tr>
<td>Bl. 10</td>
<td>63</td>
<td>99</td>
<td>250 000</td>
<td>43.4</td>
<td>168 958</td>
<td>1.48</td>
</tr>
<tr>
<td>Bl. 11</td>
<td>64</td>
<td>101</td>
<td>38 000</td>
<td>43.4</td>
<td>160 784</td>
<td>0.24</td>
</tr>
</tbody>
</table>
The damage index $D$ of the weld was evaluated as 6.73 for 1.14 million cycles, indicating a high probability for the appearance of a fatigue crack. But unfortunately no crack could be found.

However, if the more realistic 50 % survival probability is used, the fatigue strength is raised by a factor of $\frac{20}{36} = 1.94$ and the damage index drops to $\frac{6.73}{1.94^2} = 0.92$, meaning that 92 % of the fatigue life of the weld was consumed during the test.

A similar evaluation for the shear connector was not performed, mainly for the following reasons:

- The stress range is known at a section 30 mm above the main girder flange, but as the bending distribution in the shear connector is uncertain, the stress range in the most critical section is not known and can only be guessed.

- Moreover, to the best knowledge of the author, the fatigue resistance for an axially loaded shear connector, subjected to a multi-axial bending, is unknown.
6 Detailing Aspects of the Connection of Web Stiffeners to Top Flanges

In composite bridges with welds connecting vertical web stiffeners to the top flanges of the main girders, rotations of the concrete deck can induce fatigue cracks in welds connecting the stiffeners and flanges. Since welds normally have residual stresses, both positive and negative rotations can add up to the cumulative damage.

The most important factors influencing the probability for a deformation induced fatigue crack to occur at a weld between a vertical stiffener and a top flange of a main girder are:

- The ratio between the span of the concrete deck and the thickness of the deck and where a higher ratio implies larger deformations
- The position of the "slow traffic lane" relative the main girders, meaning that traffic lanes intended for the heavy trucks preferably should be placed as close as possible to a main girder
- The resistance of the steel against the rotations of the deck i.e. the stiffness and the bracing of cross girders and web-plate stiffeners

As cross girders and vertical web stiffeners are necessary in all composite bridges, it is suggested that designers should:

- **Enhance their fatigue strength**: Welds between stiffeners and main girder top flanges should preferably be butt welds rather than fillet welds.
- **Eliminate stress concentrations**: Stress concentrations at the end of stiffeners should be reduced by including a soft transition between the stiffeners and flanges.
- **Reduce the restraint**: Bracing elements near joints of interest should if possible be excluded. If that is not possible, the connections between the bracing elements and stiffeners should be designed to be as flexible as possible. Moreover, designers should pay attention to longitudinal- as well as transversal positions of shear connectors’ relative vertical web stiffeners. At sections...
where tensile force in shear connectors can be expected, it is recommended to exclude the shear connectors. If that is not an option, the shear connectors should be positioned in a narrow transversal spacing and centred above the main girder web plate.

It is left to the bridge designer to estimate the necessity of the above suggestions for every individual bridge.

The above suggestions are visualized by a few examples of design issues related to one in Sweden commonly used type of intermediate cross girder.

6.1 Detailing connections at intermediate cross girders

Even though the traffic lanes in reality are in a position centred above the main girders, the fatigue load is conservatively assumed to be travelling in the most unfavourable positions according to the figure above.

- Load position 1: the vehicle centrally in between the main girders
- Load position 2: the vehicle on the cantilever of the deck

Each of the two load positions is however checked independently of the other.
The stiffener-to-top flange connections are checked in selected points for rotations caused by a vehicle passing by the cross girder. The vehicle corresponds to fatigue load model 3, EC 1991-2, 4.6.4.

For illustrative purpose, the cross girder connections to the vertical stiffeners are designed in two different ways; see the figure and the descriptions below;

![Figure 143 Cross girder – Stiffener connection](image)

**Connection to main girder A:**
The cross girder is welded to the vertical stiffener. The connection plates between the stiffener and the top chord and diagonal of the truss are arranged with the flat side perpendicular to the vertical stiffener.

Furthermore and with the purpose to make the stiffener more flexible to deform and to eliminate stress concentrations in the top corner of the stiffener, a cut-out is introduced into the stiffener, see figure 144 below.
The above suggested geometry of the cut-out is valid only for the example in this thesis and should not, without further relevant analyses, be used in real structures.

The effects of such a cut-out as the one in figure 144 have been studied by for example Kainuma et.al [23] who performed three-point bending tests on three different kinds of web-gap plates. The specimens, which are seen in figure 145, were fabricated with a thickness of 19 mm and were welded with fillet welds of size 6 mm.
Specimen CN – No cut-out
Specimen CE – Two cut-outs of radius 15 mm
Specimen CC – One cut-out of radius 30 mm

Based upon the result of the testing, the Japanese research team made the following conclusions:

- Fatigue cracks initiate from the turn-around weld toe of web gap plate and propagate through the width of the plate. With a cut-out, the fatigue cracks initiate, not from the turn-around weld toe, but instead the fillet weld toe and propagate through the thickness of the plate. However, fatigue cracks initiates from the cut-out corner if the cut-out is not sufficiently finished. The authors stated that the possibility of fatigue cracking from the cut-out of 35 mm radius in an actual plate girder bridge seems to be small.

- When the fatigue test result were plotted against rotation-angles of the web-gap plates, the fatigue life of the plate with the cut-outs of 15 mm radius improved by approximately 3 times in comparison to the plates without cut-outs. When the cut-out of 35 mm radius was placed at the mid-height of the plate, the fatigue life improve by approximately 13 times.

In another report [25], geometric parameters as the radius of the semi-circular cut-outs and the distance from the cut-out to the top surfaces of the web-gap plate were studied by the same Japanese research team. Both finite element analyses and static loading tests were performed.

![Figure 146 Tested cut-out geometries](image)

<table>
<thead>
<tr>
<th>analytical model</th>
<th>radius of cut-out R (mm)</th>
<th>distance from upper flange L (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R15-30</td>
<td>15</td>
<td>30</td>
</tr>
<tr>
<td>R15-40</td>
<td>15</td>
<td>40</td>
</tr>
<tr>
<td>R30-45</td>
<td>30</td>
<td>45</td>
</tr>
<tr>
<td>R30-55</td>
<td>30</td>
<td>55</td>
</tr>
<tr>
<td>R30-65</td>
<td>30</td>
<td>65</td>
</tr>
<tr>
<td>R20-75</td>
<td>20</td>
<td>75</td>
</tr>
<tr>
<td>R30-75</td>
<td>20</td>
<td>75</td>
</tr>
<tr>
<td>R40-75</td>
<td>40</td>
<td>75</td>
</tr>
<tr>
<td>R50-75</td>
<td>50</td>
<td>75</td>
</tr>
<tr>
<td>R60-75</td>
<td>60</td>
<td>75</td>
</tr>
</tbody>
</table>

The authors made the following conclusions:

- A semi-circular cut-out of about 30 mm radius and placed at mid-height of a web-gap plate reduced the stress at the fillet weld toe at the upper end of the web-gap plate by approximately 50%.
- It was noticed that a stress concentration occurred at the edge of the cut-out. But since the fatigue strength of the cut-out was improved by a finishing with a grinder after the flame-cutting, the fatigue life of the web-gap plate was still around 9 times longer in comparison to a comparable plate but without a cut-out.

- A reduction of web-gap plate section due to a cut-out has no influence on the deflection of the concrete slab.

Figure 147 Elliptical cut-out geometry as evaluated by Lechner, Greiner et.al. Picture taken from [24]

Connection to main girder B:
The cross girder is bolted to the vertical stiffener. The stiffener is in turn connected at a straight angle to the top flange of the main girder. The connection is very typical and commonly used in Swedish composite Bridges.
6.6.1 Vehicle in-between the main girders

According to the finite element analysis, the effect of a more deformable connection is prominent as can be seen in the figures below and by comparing for example point A₁ with the corresponding point B₁.

The peak stress in the top corner of the stiffener connected to main girder A (position A₁) is approximately 30% of the corresponding peak stress in the stiffener connected to main girder B (position B₁).

Figure 148 Deformation of the cross girder for a load centrally between the main girders

Figure 149 Stiffener stresses in the stiffener attached to main girder A. Top chord intact
The stress maximum is as intended shifted from the outer corner of the stiffener to the inner radius of the cut-out where the fatigue class is much better.

Table 23 Actual stresses and corresponding normalized stresses achieved in FE-model for a vehicle centrally between the two main girders and with the top chord intact

<table>
<thead>
<tr>
<th></th>
<th>$\sigma_{\text{FEM}}$</th>
<th>$\sigma_{\text{FEM, Norm}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>-15,3</td>
<td>0,31</td>
</tr>
<tr>
<td>A2</td>
<td>-19,6</td>
<td>0,40</td>
</tr>
<tr>
<td>A3</td>
<td>-72,0</td>
<td>1,47</td>
</tr>
<tr>
<td>B1</td>
<td>-48,9</td>
<td>1,00</td>
</tr>
</tbody>
</table>

As connection B is considered as the standard design of similar joints in Swedish composite bridges, all stiffener stresses are normalized in the sense that they are divided by the peak stress in position B1.

$\sigma_{\text{FEM, Norm}}$ in the above table relates all achieved stress peaks in finite element models to the standard procedure joint design, i.e. to a reference value for stiffener peak stresses at position B1.
To be able to compare the effect of the notch and effect of the rotated connection plate between the top chord and the vertical stiffener, the finite element model was altered in such a way that the notch at the stiffener connected to main girder B was removed.

By performing this operation, the two stiffeners whereas one is connected to main girder A and one is connected to main girder B, becomes identical. The deformation of the cross girder is seen in the figure below.

![Figure 151 Deformation of the cross girder. Both stiffeners have no cut-out](image)

It was found that the stress peak in point A₁ and B₁ approximately was the same, meaning that the twisting of the connection plate had a negligible effect. Even though the moment of inertia is reduced as the bending of the connection plate take place around the minor axis of the plate, the length of the plate is all too short and the bracing effect is intact.

Another option to reduce the bracing effect of cross girders is to remove the top chord of the truss as soon as the concrete deck is casted. This is possible due to the fact that the benefits of the top chord are in principal negligible or at least normally very small in the service limit state. This option has been prescribed at a number of recently built composite bridges in Sweden.

In fact, composite bridge girders can even be designed and manufactured without any bracing element at all. Not even at the construction phase. However, this option demands some extra carefulness and considerations when the bridge deck is to be casted. In particular if the bridge is a curved box-girder bridge. One example is a strongly curved steel box-girder bridge situated at Kista near Stockholm Sweden.

In this particular case, horizontal forces due to declining web plates and bending moment of loading on the outside of the box was taken by reinforcement bars welded to the shear connectors on the main girders.
The effect of the removal of the bracing top chord according to the FE-model is substantial as the stress peaks in the points of interests are even further reduced.

The stiffener stress at for example position B₁ drops from -49 MPa (reference stress, figure 150) with the top chord intact, to -34 MPa with the top chord removed, a reduction of the critical stiffener stress by approximately 30%.
The same pattern is found at the stiffener connected to main girder A, i.e. the stiffener with the cut-out, where in principal all stress peaks of interest is reduced, see figure 154 and table 24 below.

Figure 154 Stiffener stresses in the stiffener attached to main girder A. Top chord removed

Table 24 Actual stresses and normalized stresses achieved in FE-model for a vehicle centrally between the two main girders and with the top chord removed

<table>
<thead>
<tr>
<th></th>
<th>$\sigma_{\text{FEM}}$</th>
<th>$\sigma_{\text{FEM, Normalized}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>-7,0</td>
<td>0,14</td>
</tr>
<tr>
<td>A2</td>
<td>-12,9</td>
<td>0,26</td>
</tr>
<tr>
<td>A3</td>
<td>-55,8</td>
<td>1,15</td>
</tr>
<tr>
<td>B1</td>
<td>-33,9</td>
<td>0,69</td>
</tr>
</tbody>
</table>
6.6.2 Vehicle on the cantilever of the deck
As bridge girders usually have web stiffeners only on one side of the web plates, supports sections excluded, the design situation is easier as the only members that transfer tensile forces into the stiffeners are the shear connectors at the top flanges of the main girders. The problem is then reduced to a matter of arrangement of shear connectors in such a way that plausible tensile forces are minimized.

That can be done by the following:

- **Longitudinal positions**: avoid shear connectors in alignment of stiffeners
- **Transversal positions**: place shear connectors centrally above the main girder web plate

![Figure 155 Stiffener stresses and deformation of the cross girder at main girder A](image)

![Figure 156 Stiffener stresses and the deformation of the cross girder at main girder B](image)

In the figure to the left (main girder A), the position of the shear connectors are more centred in comparison to the figure to the right (main girder B). The result is a stress peak in the stiffener attached to main girder A approximately 50 % of the corresponding stress peak in the stiffener attached to main girder B.

6.2 Detailing connections at support cross girders
As stated in section 2.1.2, cross girders at supports are designed to withstand lateral forces as wind, centrifugal forces, brake loading etc. and to transfer these loads to the bearings.

However, the governing load case in the Ultimate Limit State (ULS) is normally a future exchange of the bearings. While unburden the bearings, the cross girder must withstand sectional forces of magnitudes that no service limit load comes even close to cause. Moreover and during such operations, the daily temperature can vary...
significantly, with high eccentric bending moments as a result. For this purpose it is beneficial to let the steel cross girder be integrated with the concrete deck.

As a bonus, the rotations of cross girders due to deflections of the deck should be so small that any impact upon the stiffeners reasonably should be negligible.
6.3 A suggested method for estimation of stress ranges in stiffeners

A simple method for the evaluation of bending moments in vertical stiffeners subject to imposed (or forced) rotations is suggested. The method should, ideally:

- Be simple to use and preferably not involve advanced and time-consuming finite element modelling
- Give results of the same order as corresponding results obtained from advanced finite shell element analyses
- Give conservative stress results, with a reasonably wide safety margin.

Prior to the evaluation of the fatigue relevant stress range in a stiffener, a bridge designer have to make a few reasonable assumptions;

- A transverse residual stress at least of the order of one quarter of the yield stress and upward should be assumed. Because of residual stresses, the designer must assume that both positive and negative rotations add up to the cumulative damage
- As fatigue life assessment relies on testing to relate the nominal stress in a member to the number of cycles a detail can undergo at a particular stress range. The influence of local stresses is taken into account by using S-N curves obtained by testing’s of the specific detail. These empirical relations encapsulate the influence of stress raisers but require knowledge only of global stress. Accordingly, the bridge designer should assume that effects due to stress raisers already are included in the appropriate fatigue detail category
- The designer should assume that one passage of a vehicle, for example fatigue load model 3 according to EC 1991-2, 4.6.4 give arise to one fatigue relevant stress cycle. This assumption have been validated by the performed measurements on the Vårby Bridge
- As one suggestion is that the principal of “most unfavourable load position” is used, it is reasonable that a change of sign of fatigue relevant stresses are not considered. This means that all heavy vehicles should be assumed to pass by connections of interest in the same transversal position at the bridge deck

A five step procedure is described in the following;

**Step 1:** Find the most unfavourable load position

The fatigue relevant stress \( \Delta \sigma = (\Delta \sigma_{\text{max}} - \Delta \sigma_{\text{min}}) \) should be evaluated for a vehicle positioned in the most unfavourable position. Hence independently of the real position of the traffic lanes

**Step 2:** Evaluate the rotation of the deck for a vehicle in a position according to “step 1”
Having the most unfavourable position found, the next step is to evaluate the governing rotation of the deck.

As it is possible that connected bracing elements, i.e. vertical stiffeners and adjoining cross girders, are by themselves influencing the magnitude of the deck rotation, it is suggested that the maximum rotation is evaluated as if there was no stiffener or cross girder in the studied section. The magnitude of the rotation could for instance be evaluated by means of a simple finite shell element model and with the assumption of the bridge deck as simply supported upon the main girders. The approach should for all cases give a conservative nominal stress results.

**Step 3:** Impose the rotations upon a beam element frame and evaluate the governing bending moment

By imposing the rotation upon a beam element frame with appropriate boundary conditions, the governing bending moment in the cross section can be evaluated.

**Step 4:** Evaluate the effect of the weight of the vehicle

The effect of the weight of the vehicle, i.e. the axial force in the stiffener are normally small in comparison to effects of imposed rotations and can be estimated by ways of reasoning by the bridge designer.

**Step 5:** Evaluate the nominal fatigue relevant stress in the critical area

Knowing the bending moment and the axial force in the stiffener, the fatigue relevant stress can be evaluated and compared to the design criteria in accordance to the governing code.

The five step procedure is illustrated in the following section by an example.
6.3.1 Examples of evaluations of fatigue relevant stress

The example illustrates the effect of the deflection of the bridge deck upon vertical stiffeners at an intermediate cross girder at a real bridge structure, see the figure below;

Figure 159 Intermediate cross girder. Bridge crossing Forssjön

The fatigue resistance of the joint are evaluated according to EC 1993-2:2006 and EC 1993-1-9:2005 by using the so called “lambda-method”.

In [26], the equivalent constant amplitude stress range at $2 \times 10^6$ cycles is given as;

$$\sigma_{eq} = \lambda \cdot \phi \cdot (\Delta \sigma_{max} - \Delta \sigma_{min}) \quad (Eq. 39)$$

In Eq. 39, the stress range $(\Delta \sigma_{max} - \Delta \sigma_{min})$ is amplified by a dynamic factor $\phi$. But as such a factor already is included in the fatigue load model 3, it is consequently set as equal to one.

The stress range $(\Delta \sigma_{max} - \Delta \sigma_{min})$ is further amplified by a number of $\lambda$–parameters (or equivalent fatigue damage factors) which takes the following into account; span lengths ($\lambda_1$), traffic volume ($\lambda_2$), service life time ($\lambda_3$) and traffic in adjacent traffic lanes ($\lambda_4$).

The resulting equivalent damage factor $\lambda$ is given as;

$$\lambda = \lambda_1 \cdot \lambda_2 \cdot \lambda_3 \cdot \lambda_4 \quad (Eq. 40)$$
In this example, the following parameters were used:

\[ \lambda_1 = 2.55; \lambda_2 = 0.625; \lambda_3 = 0.972 \text{ and } \lambda_4 = 1.0, \text{ accordingly, } \lambda = 1.55 \]

Having an accurate value of \( \lambda \) established, the first step is to evaluate the most unfavourable load position, i.e. the vehicle position that induces the largest rotation at sections of interest. As the particular bridge consists of two main girders, the most unfavourable position is either as far out on the cantilever as possible or somewhere in-between the two main girders. In this example assumed as centrally in-between the main girders.

The second step is accordingly to evaluate the maximum and minimum rotations of the deck based upon the result of step 1. This can easily be done in for example a finite element model comprising only of a continuously and simply supported deck.

![Figure 160 Rotation of the deck with vehicle centred between the main girders](image)

The maximum- and minimum rotations are seen in the table below;

<table>
<thead>
<tr>
<th></th>
<th>Girder 1</th>
<th>Girder 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load in between the main girders</td>
<td>-0.43</td>
<td>0.43</td>
</tr>
<tr>
<td>Load at the cantilever of the deck</td>
<td>0.35</td>
<td>-0.12</td>
</tr>
</tbody>
</table>
As both positive and negative rotations are assumed to add up to the cumulative damage, the most unfavourable position of the vehicle is assumed to be in between the two main girders and is hence given the highest priority.

**Load in-between the two main girders**

The bending moment at the top of the vertical stiffener can be evaluated in a simple beam element frame, see the figure below.

![Figure 161 Boundary- and load conditions of the beam element frame model](image)

The following boundary conditions are adopted:

**Support 1 and 2**

The supports are fixed for rotations around the Y-axis. Moreover, the supports are free to translate along the Z-axis but elastic along the X-axis. The elastic spring coefficient can be evaluated based upon the contraction of a prismatic concrete member and axially loaded by a unit load.

**Support 3 and 4**

The supports are free to rotate around the Y-axis. The supports are elastic for translation in the direction of the X-axis and rigid for translation along the Z-axis. The elastic spring coefficient can be evaluated based...
upon the out-of-plane deflection of the bottom flanges, loaded perpendicular its length by a unit load.

The nodes corresponding to support 1 and 2 are then imposed by the rotation $\pm \theta$ around the Y-axis (but of opposite sign in each node).

The bending moment in the stiffener at a section equal to the upper short side of the stiffener is evaluated for two separate cases:

- Case A: Cross girder with the top chord intact
- Case B: Cross girder with the top chord removed

The result (bending moment) for both cases can be seen in figure 162 and figure 163 below.

**Figure 162 Bending moment in the stiffener with the top chord intact (Case A)**

- $M = 13.3$ KNm (Case A)

It is seen in the above figure that the bracing effect of the upper strut (top chord) is substantial.
As is seen, the effect of the removal of the top chord is substantial as the bending moment for Case B is only 54% of the corresponding bending moment for Case A. It should however be noticed that the bending moment, for both alternatives, is evaluated for a rotation of same magnitude. In a real structure and when dismissing the bracing effect of the top chord, the cross section will consequently become less reluctant to bend why the operation most likely would infer larger rotations and a higher governing bending moment.

Having the bending moment evaluated, the 4th step remains i.e. the evaluation of the impact of the weight of the vehicle.

As the distance between the two bogies for the vehicle in fatigue load model 3 is 7.2 m, it is reasonable to assume that only the weight of one bogie (of a total of two) affects the connection. Moreover, as the vehicle is in a position far away from the connections of interest, the weight should reasonably be spread wider through the bridge deck and therefore be carried by a large portion of the main girder web plates.

It is also assumed, considering the centrally position of the vehicle that ½ of the weight of the bogie is transferred to each one of the two main girders. The cross section will thus carry 0.5·240 = 120 KN distributed on an effective area consisting of a strip of the web plate of a width equal to 2.0 m and the area of the stiffener.

In a real structure and for the current load position, the participating width of the main girder web plate should by all reasoning be larger.
Table 26 Evaluation of the axial stress contribution to the fatigue relevant stress range

<table>
<thead>
<tr>
<th></th>
<th>Width</th>
<th>Thickness</th>
<th>Area</th>
<th>Proportion</th>
<th>N</th>
<th>$\Delta\sigma_{axial}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Web plate</td>
<td>[mm]</td>
<td>[mm]</td>
<td>[mm²]</td>
<td>[-]</td>
<td>[KN]</td>
<td>[MPa]</td>
</tr>
<tr>
<td>2 000</td>
<td>20</td>
<td>40 000</td>
<td>0.90</td>
<td>108</td>
<td>2,7</td>
<td></td>
</tr>
<tr>
<td>Stiffener</td>
<td>180</td>
<td>4 500</td>
<td>0.10</td>
<td>12</td>
<td>2,7</td>
<td></td>
</tr>
<tr>
<td>Σ</td>
<td></td>
<td>44 500</td>
<td>1,00</td>
<td>120</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The axial stress range $\Delta\sigma_{axial}$ superposes the bending stress range according to Navier.

For reasons of simplicity, the bending resistance of the cross section is evaluated based upon a participating strip of the main girder web plate of 500 mm ($= 25 \cdot t_w$) and of the cross section of the stiffener ($25 \cdot 180 \text{ mm}^2$).

Hence the bending resistance of the cross section is:

$$W_{\text{stiffener}} = 273 756 \text{ mm}^3$$

All the necessary parameters for the last step, i.e. the evaluation of the fatigue resistance for the connection are now known and are consequently conducted in table 27 below. It should in this context be noticed that the calculated stresses in the table are linear elastic stresses, calculated as if no stress concentration exists. In reality stress concentrations exists, mainly due to an abrupt change of cross sectional area and due to the geometry of the weld itself.

As previously stated, is suggested that a bridge designer should assume that effects of existing stress concentrations are included in the appropriate fatigue detail category, FAT 36/40 or FAT 71, EC 1993-1-9:2005. Preferably FAT 71 as in this example as another suggestion in this thesis is that welds between stiffeners and top flanges should be performed as butt welds and not fillet welds.

The fatigue resistance is expressed as a utility ratio, $U$, where the design criterion is that $U$ should be $\leq 1.0$.

Table 27 Evaluation of the fatigue resistance of the stiffener-to-top flange connection

<table>
<thead>
<tr>
<th>Case</th>
<th>$M$ [KNm]</th>
<th>$\Delta\sigma_{bend}$ [MPa]</th>
<th>$\Delta\sigma_{axial}$ [MPa]</th>
<th>$\Delta\sigma_{E,2}$ [MPa]</th>
<th>$\Delta\sigma_{E,3}$ [MPa]</th>
<th>$U$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>13,3</td>
<td>48.6</td>
<td>2.7</td>
<td>79.5</td>
<td>52.6</td>
<td>1.51</td>
</tr>
<tr>
<td>B</td>
<td>7,2</td>
<td>26.3</td>
<td>2.7</td>
<td>45.0</td>
<td>52.6</td>
<td>0.85</td>
</tr>
</tbody>
</table>

An explanation to the above table is given below;

$M$ Bending moment at the top of the stiffener due to the imposed rotation $\theta$
and evaluated in the beam element frame models

\[ \Delta \sigma_{\text{bend}} \quad \text{Maximum nominal bending stress in the top corner of the stiffener due to the bending moment } M \text{ and evaluated as: } \Delta \sigma_{\text{bend}} = \frac{M}{W_{\text{stiffener}}} \]

\[ \Delta \sigma_{\text{axial}} \quad \text{Axial stress in the stiffener} \]

\[ \Delta \sigma_{\text{E,2}} \quad \text{Equivalent stress range: } \Delta \sigma_{\text{E,2}} = \lambda \cdot (\Delta \sigma_{\text{bend}} + \Delta \sigma_{\text{axial}}) \]

\[ \Delta \sigma_c \quad \text{Fatigue strength including the partial safety factor for fatigue strength } \gamma_{\text{Mf}} = 1.35 \Rightarrow \Delta \sigma_c = \frac{71}{1.35} \]

\[ U \quad \text{Utility ratio defined as: } U = \frac{\Delta \sigma_{\text{E,2}}}{\Delta \sigma_c} \]

It is seen that only Case B meets the design criteria \( U \leq 1.0 \). For this reason and in this particular case, the top chords of all intermediate cross girders were prescribed to be removed after the bridge deck was casted.

To further improve the fatigue resistance of the joint and with a purpose to eliminate stress concentrations in the turn-around weld at the top corner of the stiffener, the width of the stiffener was locally increased near the top flanges – from 180 mm to 230 mm, see the figure below.

![Figure 164 Stiffener connection to the top flange at Bridge 4827-1, crossing the lake of Forssjön](image)

It is questionable if this stiffener design option is more optimal than semi-circular cut-outs which were under discussion in section 6.1 but it is believed that it is superior in comparison to the straight angle connection.
The design also have one advantage in comparison to semi-circular cut-outs; a less probable risk of transferring the problem from the weld to the stiffener material. It is therefore recommended that cut-outs are not used as a standard detailing option, but instead more as a last resort in cases when dealing with connections with extremely high fatigue relevant stresses. It is also recommended that a use of cut-outs should be preceded by finite element studies on the optimizing of the detail to find the optimal geometry to minimize the secondary stresses at the welds as well as cut-outs.

**Load at the cantilever of the concrete deck**

If the governing rotation is such that stiffener tensile forces are expected, the evaluation of the fatigue relevant stress range becomes more complicated as the response of the steel is depended on both the longitudinal- as well as transversal arrangement of shear connectors on the main girder top flanges.

A bridge designer has to consider two cases, C & D, as listed below:

- Case C: no shear connectors in alignment with the stiffener
- Case D: a shear connector in alignment with the stiffener

It is in this thesis suggested that shear connectors in alignment with stiffeners should be avoided (i.e. in accordance with Case C). This option has mainly two large advantages:

1. Some “redistribution” of tensile force will occur, mainly due to the elongation of the shear connectors and the deformability of the flanges for loading perpendicular the plane of the flanges.

   By exclusion of shear connectors in alignment with stiffeners, the joints become more flexible and the global bending moment in the stiffeners decreases. The stiffeners and cross girders will not experience rotations of equal magnitude as the deck slab, as gaps will open up between the deck and the top flanges. This is seen when studying the result of the LVDT-device used during the monitoring of the Vårby Bridge. Gap-openings were also observed at the measurements performed by the Austrian investigation team.

2. With a shear connector in alignment with a vertical stiffener, the global bending moment in the stiffener will be larger. Furthermore, as the tensile force is transferred through the shear connectors, in this case mainly through the particular shear connector in alignment with the stiffener, a more “local effect” will occur as the force will be introduced to the stiffener at a single and discrete point, see the figure below.
This is not advantageous for either the weld between the stiffener and the top flange.

Nevertheless, if the nearest shear connectors are not extremely far away from the stiffener, some amount of tensile force will unavoidably be transferred into the stiffener. To quantify the amount, the bridge designer must consider:

- The axial stiffness of the shear connectors and
- The deformability of the steel top flanges for loads at various positions, acting in a direction perpendicular to the plane of the flanges

There are other flexibilities of influence but they are for reasons of simplicity not included here.

There are indeed difficulties to reduce this highly three dimensional problem into a plane one, but an attempt to do so is made in the following where the flexibilities are treated as a system of elastic springs in series.

\[
\frac{1}{S_{\text{total}}} = \frac{1}{S_{\text{flex}}} + \frac{1}{S_{\text{cross girder}}}
\]  

(Eq. 41)

The cumulative spring stiffness \(S_{\text{flex}}\), considering both the axial stiffness of the shear studs and the bending stiffness of the flanges is evaluated based upon the result achieved in a miniature finite element model, see figure 166 and 167.

\(S_{\text{flex}}\) represents the combined effect of the two flexibilities mentioned earlier. The miniature finite element model consists of a small part of the flange i.e. the part of the top flange located on the inside of the web girder web plate. In addition, a small part of the stiffener is also modelled as well as four pair of shear connectors, see figure 166 below.
The edge of the flange connected to the main girder web plate is set to be fully restrained for all three translations and all three rotations. The remaining edges of the flange are not supported at all.

All stiffener edges except the very edge attached to the flange are free to rotate and translate in all directions.

![Diagram](image)

*Figure 166 Plan view of the part of the flange modelled in a small finite element model*

In the finite element model and for a vertical unit load acting on the stiffener, the vertical displacement is evaluated at the same position as the position of the load and for the following seven cases:

1. No shear connector have vertical support
2. Only shear connector pair number 1 have a vertical support
3. Only shear connector pair number 2 have a vertical support
4. Only shear connector pair number 3 have a vertical support
5. Only shear connector pair number 4 have a vertical support
6. All pair of shear connectors have vertical supports
7. All pair of shear connectors have vertical supports + one additional shear connector placed in alignment of the stiffener and the unit load

When evaluating for example the stiffener displacement for the 1st pair of shear connectors only the nodes at the positions of the studied pair are set to be fixed for translation in the vertical direction. The unit load is applied in an appropriate node and the vertical displacement in the same node is registered.

The procedure is repeated for the rest of the cases. A complete result is seen in table 28.
Figure 167 Part of local finite element model with shear connector pair number 1 modelled as supports in the vertical direction

Table 28 Evaluation of the system flexibility due to the deformability of the top flanges

<table>
<thead>
<tr>
<th>Case</th>
<th>Vertical support</th>
<th>( x ) [mm]</th>
<th>( P ) [KN]</th>
<th>( \delta ) [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>No shear connector</td>
<td>(-)</td>
<td>100</td>
<td>0,52</td>
</tr>
<tr>
<td>2</td>
<td>Only pair 1</td>
<td>150</td>
<td>100</td>
<td>0,38</td>
</tr>
<tr>
<td>3</td>
<td>Only pair 2</td>
<td>300</td>
<td>100</td>
<td>0,50</td>
</tr>
<tr>
<td>4</td>
<td>Only pair 3</td>
<td>450</td>
<td>100</td>
<td>0,52</td>
</tr>
<tr>
<td>5</td>
<td>Only pair 4</td>
<td>600</td>
<td>100</td>
<td>0,52</td>
</tr>
<tr>
<td>6</td>
<td>All pairs</td>
<td>(-)</td>
<td>100</td>
<td>0,38</td>
</tr>
<tr>
<td>7</td>
<td>All pairs + one connector in alignment of the stiffener</td>
<td>(-)</td>
<td>100</td>
<td>0,15</td>
</tr>
</tbody>
</table>

\( x = (\pm) \) Longitudinal distance (distance along the flange), from the stiffener to each pair [mm]

\( P \) Unit load 100 KN

\( \delta \) Vertical displacement of the stiffener at a point at the stiffener and in alignment of the shear connector
It is seen that the vertical deformation $\delta$ for pair number 2 to number 4 was found to be approximately same as if no shear connectors were present. The pairs were consequently assumed to be of minor influence.

For Case C and as all other pair of shear connector except the 1st pair is of no influence, the cumulative elastic stiffness $S_{flex}$ could simply be calculated as:

$$S_{flex, case C} = \frac{P}{\delta} = \frac{100}{0.38} = 263 \text{ KN/mm}$$  \hspace{1cm} (Eq. 42)

The above described procedure was then repeated for Case D, i.e. the case with a shear connector in alignment of the vertical stiffener. The response of the system was found to be more rigid as the vertical deformation $\delta$ was 0.15 mm. The cumulative elastic stiffness $S_{flex}$ for Case D could accordingly be calculated as:

$$S_{flex, case D} = \frac{P}{\delta} = \frac{100}{0.15} = 667 \text{ KN/mm}$$  \hspace{1cm} (Eq. 43)

As the elastic spring stiffness $S_{flex}$ simulating the combined effect of the elongation of the shear connectors and the out-of-plane deformation of the flanges is known, it is still left to evaluate the elastic spring stiffness of the cross girder itself. This is done by the introduction of a vertical unit load (100 KN) upon the cross girder and the evaluation of the vertical displacement of the cross girder at a section of most interest, see the figure below.

![Deformation evaluation](image)

Figure 168 Deformation of the cross girder for a unit load 100 KN. The unit load is in the same position as when evaluating the spring stiffness due to the deformability of the flanges.
At the point indicated in the above figure, the deformation for a unit load of 100 KN was found to be 0,9 mm. The elastic spring stiffness could accordingly be calculated as:

\[ S_{\text{cross girder}} = \frac{P}{\delta} = \frac{100}{0.9} = 111 \text{ KN/mm} \]  
(Eq. 44)

Having the flexibilities of the system evaluated, it was assumed that the total rotation, \( \theta \), can be expressed as the sum of two parts; one part depended upon the flexibility \( S_{\text{flex}} \) and one part depended upon the flexibility \( S_{\text{cross girder}} \). Hence:

\[ \theta = \theta_{\text{flex}} + \theta_{\text{cross girder}} \]  
(Eq. 45)

Accordingly if \( \theta \) is set to 0:

\[ S_{\text{flex}} \cdot \theta_{\text{flex}} = S_{\text{cross girder}} \cdot \theta_{\text{cross girder}} \]  
(Eq. 46)

Some re-arranging gives:

\[ \theta_{\text{cross girder}} = \frac{S_{\text{flex}} \cdot \theta_{\text{flex}}}{S_{\text{cross girder}}} \]  
(Eq. 47)

Combining eq.45 and 46 gives:

\[ \theta_{\text{flex}} = \theta - \frac{S_{\text{flex}} \cdot \theta_{\text{flex}}}{S_{\text{cross girder}}} \]  
(Eq. 48)

By dividing all terms in eq. 48 with \( \theta_{\text{flex}} \) and by some additional rearranging, the relative portion \( \theta_{\text{rel}} \) of the total rotation \( \theta \), and deduced to the elongation of the shear connectors and the out-of-plane deformation of the flanges can be estimated by the following expression:

\[ 1 = \frac{\theta}{\theta_{\text{flex}}} - \frac{S_{\text{flex}}}{S_{\text{cross girder}}} \Rightarrow \theta_{\text{flex}} = \frac{\theta}{1 + \frac{S_{\text{flex}}}{S_{\text{cross girder}}}} \]  
(Eq. 49)

The resultant rotation, \( \theta_{\text{res}} \), i.e. the rotation eventually to be imposed on a simple beam-element frame model, can be evaluated as:

\[ \theta_{\text{res,Case c}} = \theta \cdot \left( 1 - \frac{1}{1 + \frac{S_{\text{cross girder}}}{S_{\text{flex,Case c}}}} \right) = \theta \cdot \left( 1 - \frac{1}{\frac{1}{1 + \frac{263}{111}}} \right) \]  
(Eq. 50)
\[ \theta_{res,\text{Case } C} = 0.70 \cdot \theta \]

And for case D,
\[ \theta_{res,\text{Case } D} = \theta \cdot \left( \frac{1}{1 + \frac{S_{flex,\text{Case } D}}{S_{\text{Cross girder}}}} \right) = \theta \cdot \left( 1 - \frac{1}{1 + \frac{667}{111}} \right) \quad \text{(Eq. 51)} \]

\[ \theta_{res,\text{Case } D} = 0.86 \cdot \theta \]

For Case C, it was found that 30 % of the total rotation can be deduced to the flexibility \( S_{flex} \), why the rest of the flexibility, 70 %, is deduced to \( S_{\text{Cross girder}} \).

Similarly, in Case D it was found that 14 % of the total rotation can be deduced to the flexibility and the rest of the flexibility, 86 %, is deduced to \( S_{\text{Cross girder}} \).

The rotations \( \theta \) could then, in the same manner as for the case with the load in-between the two main girders, be collected in a simple finite element model and reduced to resultant rotations \( \theta_{res} \).

\[ \theta_{res} \] can then be imposed on the same beam element frame as in the previous example (a load in-between the main girders).

The result (bending moment) for case C, i.e. the case with no shear connector in alignment of the vertical stiffener is seen in the figure below:

---

Figure 169 Rotation of the deck with vehicle centred between the main girders

---
For this particular case and as the weight of the vehicle in fact is of benefit for the joint, the axial force in the cross section is assumed to be of zero value. It should however be noted that the assumption, for a real design case, might be all too conservative.

The fatigue resistance of the joint, expressed as a utility ratio $U$, can finally be evaluated.

<table>
<thead>
<tr>
<th>Case C</th>
<th>$M$</th>
<th>$\Delta \sigma_{\text{resid}}$</th>
<th>$\Delta \sigma_{\text{axial}}$</th>
<th>$\Delta \sigma_{\text{E}}$</th>
<th>$\Delta \sigma_{\text{C}}$</th>
<th>$U$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[KNm]</td>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[-]</td>
</tr>
<tr>
<td>Case C</td>
<td>6,1</td>
<td>22,3</td>
<td>0</td>
<td>34,5</td>
<td>52,6</td>
<td>0,66</td>
</tr>
</tbody>
</table>

It is seen in the above table that the design criterion $U \leq 1,0$ is fulfilled even with the upper strut (top chord) intact.
7 Discussions, Conclusions and Suggestions for Future Research

7.1 Discussions and answers to the research questions

In section 1.1, a few research questions were asked. In this chapter, an attempt to answer the questions is made. The chapter ends with a few suggestions aimed for future researchers.

1) What is the true cause of the cracks in Vårby Bridge?

The results of the investigations presented in this thesis unambiguously indicate that material fatigue is the cause of the cracks. All other possible causes, with varying degrees of plausibility, can be excluded since:

- Striations and so-called "beach-marks" were found at the fracture surface of a crack in a small sample of a weld collected from the bridge. Striations and beach-marks are both strong indications of crack growth through fatigue.
- The frequency and magnitude of fatigue-relevant stresses in areas of interest were found to be sufficient to explain the cracking. Indeed, such stress peaks are numerous since the traffic flows are heavy, and the passage of every sufficiently heavy vehicle could contribute to fatigue damage.
- Evaluated fatigue life of two of the joints and performed on basis of the long-term monitoring, using a more realistic 50 % survival probability for FAT 36 ($\Delta\sigma_C = 70$ MPa) were found to be 7 respectively 14 years. The result seems reasonable considering the fact that the cracks were discovered 10 years after the bridge was taken into service.
- Performed finite element analyses indicate that high fatigue relevant stresses occur in areas of interest.
Discussion and Suggestions for Future Research

The fatigue-relevant stresses at the origin of the cracks are believed to be due to imposed (or forced) rotations of rigid cross girders and web stiffeners. Such rotations occur when a slender concrete deck deflects as heavy vehicles pass.

Moreover, as most stress peaks at points of interest were found to be compressive, and fatigue cracks usually form and grow in areas where the resultant stress is tensile, the presence of tensile residual stresses at the origin of the cracks is assumed. Further, the direction of the residual stress field is assumed to be perpendicular to the crack planes. Research results supporting both assumptions have been presented in the thesis. It is also believed that the mean stress in fatigue-relevant stress cycles imposes high tension due to high residual stress at weld toes adding to the normally nominal compressive stress caused by live loads.

2) Why did the cracks appear at this particular bridge?

Several adverse factors influence Vårby Bridge. The most substantial are listed and described below:

- **Large rotations of the bridge deck**
  
  As the bridge deck is wide – 14,0 m - and carried by two main girders only; the distance between the girders is consequently also wide. Moreover; to be able to resist governing shear forces in the concrete deck, the thickness of the deck is > 340 mm at a section above the main girders. But as the deck is tapered, the corresponding thickness at a section in the centre between the two main girders is only 280 mm. The thickness of the deck must consequently be considered as being extreme in the sense that such thin deck plates usually are found only at smaller composite bridges where the distance between the main girders are in the range of perhaps 3 m.

  Due to the combination of the large distance between the main girders and the relatively thin deck plate, the ratio between the span length of the deck and the deck thickness is large. A large ratio (or slenderness) implies large deflections when heavy vehicles pass, and consequently large rotations.

- **Large restraint**
  
  Both the vertical stiffeners and intermediate cross girders have greater bending stiffness than is appropriate for such members. The main reason for that is that the cross girders were designed to support the scaffolding when the bridge deck was cast. Furthermore, and for the same reason, the distance between the top surface of the main girders and the corresponding top surfaces of all cross girders is short, just 400 mm.
The cross-sections of interest are consequently short and stiff, which is a particularly poor combination for structural members exposed to large, cyclic rotations.

- **Inadequate fatigue strength**

  The secondary effects discussed in this thesis were not anticipated during the design of the bridge. Consequently, the cracked welds are all fillet welds rather than butt welds, which have greater fatigue strength. Furthermore, the welds were only designed to withstand shear force, thus the fillet welds are only half as wide as the stiffeners.

- **Extremely high traffic**

  By Swedish standards, Vårby Bridge carries extremely heavy traffic. Each of the two adjacent bridges carries more than 40,000 vehicles daily, including numerous heavy vehicles, all of which may contribute to fatigue damage.

  For example, during the long-term measurement campaign (104 hours in total) approximately 11 700 peaks > 10 MPa were registered. Extrapolated over a period of 120 years — the expected life time of the bridge — the corresponding number of peaks > 10 MPa exceeds 100 million.

  In future, for similar joints of bridges carrying extremely heavy traffic, use of design criteria equal to the intersection of the fatigue strength curve (m = 5) at $10^8$ cycles should be considered.

3) **How will the cracks develop, and is there any immediate hazard concerning the safety and durability of the bridge?**

   The cracks are believed to constitute no immediate hazard for the safety and sustainability of the bridge. However, in a worst case scenario, if the propagation continues, the stiffeners could in time become separated from the top flanges. If so, out-of-plane bending of the main girder web plates could occur, leading to “moustache cracks” in the main girder web plates. This would be highly undesirable and must be prevented.

   The cracking in Vårby Bridge is believed to be due to deflection of the concrete deck. This implies that the cracking is governed more by displacement than by load. In contrast to a case where cracking is governed by load, the stress intensity factor — a critical parameter for the crack growth rate — should decrease as the length of cracks increases, thereby retarding the crack growth. It is therefore likely that the propagation of the observed cracks will slow and eventually stop.

   It should be noted that the assumption that the cracking will slow has not been tested, by mechanical fracture analysis for example. However, during the installation
Discussion and Suggestions for Future Research

of the monitoring equipment, a crack was selected and marked to allow its further growth to be followed. The author strongly recommends that the bridge owner continues to inspect the crack, tentatively once a year. Furthermore, a corrosion protection should be considered.

4) Can stiffener stresses be predicted by Finite Element analysis or manually? How should bridge designers estimate fatigue-relevant stresses?

As part of the project this thesis is based upon, attempts were made to reproduce measured stiffener stresses. The structure comprising the deck and steel plates was modelled by shell elements, and the elements corresponding to the deck plate were connected to the steel girder elements by rigid links. This is equivalent to introducing rigid compatibility conditions with respect to selected displacements in chosen nodes. The stiffeners were hence constrained to follow the displacements of the deck. It was found that the model captured the global behaviour of the bridge satisfactorily. However, the modelled responses to stiffener stress peaks in critical areas were too stiff, yielding substantially higher than measured stiffener stresses.

It has been found that the shrinkage possibly can cause gaps between the deck and the steel top flanges at locations of vertical stiffeners. Such gaps have been found at cross girder-connections in the Vårby Bridge and other similar bridges. It is suggested that since heavy vehicles pass between the two main girders such gaps could in fact reduce negative nominal stresses at the top corners of stiffeners due to the rotation imposed on them. When attempts were made to reproduce such gaps in the FE-analyses, better concurrence with the measurements was found.

The Vårby Bridge was also modelled as part of a master’s project during the spring of 2010 by two students at Chalmers University of Technology under the supervision of Docent Mohammad Al-Emrani. The results were in many respects similar to those described above, with stiffener stresses of similar magnitudes.

In the investigation of fatigue cracks in Austrian bridges mentioned in section 3.4, more sophisticated analyses were performed, including modelling based on volumetric elements. Contact pairs were used to model the connections between the concrete slab and steel top flange. The research team concluded that the sheer number of parameters influencing the behaviour of this structural detail is a source of difficulties, and comparison of numerical results with field measurements can only be moderately satisfactory. Generally it was found that, due to differences in structural detailing conditions of the stiffeners, a broad band of stresses may appear at the stiffener-to-top flange connections in a single bridge.

All three analyses mentioned above have one feature in common; the finite element models yield more rigid responses than field measurements. One conclusion is thus
that future advanced finite element analyses of connections of interest probably will overestimate the stress range unless corrections are made.

Based on the results of the shell finite element analyses and measurements, attempts to reduce this complex and highly three-dimensional problem into a plane problem are being made. A simple method for evaluating bending moments in vertical stiffeners subject to imposed (or forced) rotations is suggested. The method should ideally:

- Be simple to use and preferably not involve advanced, time-consuming finite element modelling
- Give results of the same order as corresponding results obtained from advanced finite shell element analyses
- Give conservative stress results, with a reasonably wide safety margin.

5) Can bridge designers take measures against the phenomenon that caused the cracks?

Based on the analysis of the cracks in Vårby Bridge and the lab tests at LTU, it is suggested that designers of future steel and composite bridges should take the following in consideration:

- **Enhance their fatigue strength**: Provided a weld area equal to the cross-sectional area of the stiffener, the welds between stiffeners and main girder top flanges should preferably be butt welds rather than fillet welds. This simple measure should raise the fatigue strength by a factor of two and prolong the fatigue life of the weld by a factor of eight.

- **Eliminate stress concentrations**: Stress concentrations at the end of stiffeners should be reduced by including a soft transition between the stiffeners and flanges.

- **Reduce the restraint**: Bracing elements near joints of interest should, if possible, be excluded. If that is not an option, the connections between the bracing elements and stiffeners should be as flexible as possible.

It is also suggested that attention should be paid to both the longitudinal and transversal positions of shear connectors relative to vertical web stiffeners. In order to avoid the introduction of tensile forces into main girders at locations of vertical stiffeners, shear connectors should be positioned in pairs with a narrow transversal spacing. Moreover, shear connectors should not be directly aligned with vertical web stiffeners.
Discussion and Suggestions for Future Research

It is left to the bridge designer to estimate the necessity of the above suggestions for every individual bridge.

The impact of the above suggestions are visualized by a few examples of design issues related to one in Sweden commonly used type of intermediate cross girder.

7.2 Suggestions for future research
One suggestion to future researchers is to precede finite element studies calibrated by tests on the optimizing of e.g. semi-circular cut-out details and with the objective to find the optimal geometry to minimize secondary stresses. Furthermore, the tests could indicate the fatigue resistance when the maximum stress peak is transferred from the turn-around weld at the stiffeners free edge, towards the web of the main girder.

Moreover, since it is believed that the lower than expected compressive stress level at the Vårby Bridge can be explained by gaps caused by shrinkage and by some kind of deterioration of concrete due to a repeated cyclic loading, it would be of great interest to monitor a bridge, starting immediately after the final casting sequence and further on.

As it is believed that the most frequent load case for welds between stiffeners and top flanges in fact is compressive loading, and as in principal all fatigue testings’ of cruciform fillet welded joints were performed by a use of tensile loading, it is questionable if the fatigue strength FAT 36 in accordance with EC 1993-1-9:2005 is relevant. The above fatigue class presupposes a crack at the weld roots, but as is seen at the Vårby Bridge, the cracks are initiated at weld toes. A suggestion for further research work is thus the evaluation of the fatigue strength for cruciform fillet weld joints but under compressive loading.
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Appendix A - Rain Flow Counting histograms. The Vårby Bridge

Histogram for Channel 1

Histogram for Channel 2

Histogram for Channel 3

Histogram for Channel 4
Appendix A

Histogram for Channel 5

Histogram for Channel 6

Histogram for Channel 7

Histogram for Channel 8

Histogram for Channel 9

Histogram for Channel 10
Appendix A

Histogram for Channel 17

Histogram for Channel 18

Histogram for Channel 19

Histogram for Channel 20

Histogram for Channel 21
Appendix B - Cross Girders from All over the World

Austria
Appendix B

Austria

USA, State of New York
Appendix B

USA, State of New York.

END DIAPHRAGM-TYPE 3
FOR SKEWS 20° & UNDER

USA, State of New York

INTERMEDIATE DIAPHRAGM-TYPE 2

USA, State of New York
INTERMEDIATE DIAPHRAGM - TYPE 1
(FULL DEPTH)

USA, State of New York

INTERMEDIATE DIAPHRAGM-TYPE 3

USA, State of New York
INTERMEDIATE DIAPHRAGM-TYPE 2
(BOTTOM LATERAL BRACING NOT SHOWN)
USA, State of New York

INTERMEDIATE DIAPHRAGM-TYPE 3
(BOTTOM LATERAL BRACING NOT SHOWN)
USA, State of New York
Appendix B

USA, State of Texas

**TYPE ED1 THRU ED4 END DIAPHRAGMS**

For Rolled Beams and for Plate Girders with web depths less than 52". Not for use with Thickened Slab Ends shown on SB15 standard.

USA, State of Texas

**TYPE ECF END CROSS-FRAME**

For Plate Girders with web depths of 52" to 96". Not for use with Thickened Slab Ends shown on standard SB15.

USA, State of Texas
Appendix B

USA, State of Texas

TYPE D1 THRU D5 DIAPHRAGMS
For straight Rolled Beams and for straight Plate Girders with web depths less than 52". For all locations, including end bearings when Thickened Slab Ends, shown on standard SBTS, are used. Minimum stiffener width is 7" for use with these diaphragms.

USA, State of Texas

TYPE ED1 THRU ED4 END DIAPHRAGMS
For straight Rolled Beams and for straight Plate Girders with web depths less than 52". Not for use with Thickened Slab Ends, shown on standard SBTS. Minimum stiffener width is 7" for use with these diaphragms.

USA, State of Texas
Appendix B

USA, State of Texas

TYPE XF1 THRU XF3 CROSS-FRAMES
For Plate Girders with web depths of 52" to 96". For all locations, including end bearings when Thickened Slab Ends, shown on standard SBTs are used. Minimum stiffener width is 8" for use with these cross-frames.

USA, State of Texas

TYPE KF1 THRU KF3 CROSS-FRAMES
For Plate Girders with web depths of 52" to 96". For all locations, including end bearings when Thickened Slab Ends, shown on standard SBTs are used. Minimum stiffener width is 8" for use with these cross-frames.

USA, State of Texas
Appendix B

USA, State of Texas

TYPE EF END CROSS-FRAME

For Plate Girders with web depths of 52" to 96". Not for use with thickened slab ends, shown on standard SETs.
Minimum stiffener width is 8" for use with this cross-frame.

USA, State of Texas

TRANSVERSE SECTION

USA, State of Texas

ELEVATION TYPE A
USA, State of Texas
USA, State of Texas

USA, State of Texas
Appendix B

Sweden. Bridge over Faxälven at Edsele.

Sweden. Bridge over Faxälven at Edsele.

Sweden. Bridge crossing Indalsälven at Sörån.
Appendix B

Sweden. Bridge crossing Dalälven.

Bridge crossing Voxnan at Finnstuga.
Sweden. Bridge crossing Indalsälven.
Sweden. Bridge crossing Österdalälven.

Sweden. Bridge crossing Pitsundet.
Appendix B

Sweden. Bridge crossing Vindelälven.

Sweden. Bridge crossing Vindelälven.
Bridge in Borlänge, Sweden.
Appendix B

Bridge crossing Öre älv.
German cross girder. Published by kind permission of SSF Ingenieure AG

German cross girder. Published by kind permission of SSF Ingenieure AG
German diaphragm. Published by kind permission of SSF Ingenieure AG
German cross girder. Published by kind permission of SSF
Ingenieure AG

Ingenieure AG
German cross girder.

German cross girder.
German cross girder.

German cross girders.
Appendix B

Czech Republic