LICENTIATE THESIS

I-girder Composite Bridges with Lateral Bracing

Improved load distribution

Victor Vestman

Structural Engineering

LULEÅ UNIVERSITY OF TECHNOLOGY

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Cover image: Kaitainen bridge, Taivassalo, Finland. (Formatted/changed picture from source: https://commons.wikimedia.org/wiki/File:Kaitainen_bridge_1.jpg#/media/File:Kaitainen_bridge_1.jpg)

Academic thesis

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Assistant supervisor:	Adj. Prof. Dr. Robert Hällmark, Luleå University of Technology

Preface

This thesis is a big checkpoint on my journey as Ph.D Student, which started in 2020. However, my journey in the world of bridges started already in 2015. I was a young student, eager to get out from the master studies and the academic world, into work life in the industry. Everyone who is at the end of completing a course, an assignment or big project knows that the closer you are to the goal, the higher is the need of feeling finished. The same was it for me at the end of 2015, almost at my goal to being out in the industry, but where should I go? I started my career as a bridge designer at a local design office in Luleå and I was hooked from the start.

During my first years in the industry, I had the privilege to have great supervisors, colleagues, and friends in one place. For that I am forever grateful, and I hope that I in the future will participate to create the same feeling for young engineers. So, thanks for pushing me forward and for giving me the best start I could imagine in the world of bridges. One of these persons has later become a dear friend of mine and he has also inspired many other young students to start their journey as bridge designers. This friend has showed me both sides, the creative and interesting part of being a designer, and the inspiration of being a researcher with the possibility to gather, develop and implement ideas on real bridges. Thanks Prof. Peter Collin for being there for me, and I hope our journey together will continue for a long time.

Also, thanks to my supervisor Adj. Prof. Robert Hällmark, who always is able to give 110 % in his commitments. For his time and encouragement during periods where my enthusiasm for the work has been low.

To the rest of my colleagues at the Division of Structural and Fire Engineering I would like to say, love to see you more. A truly passionate group of people, whose spirits glows trough the wall of the university. To Erik Andersson and Mats Petersson, two of the main cogs in our team, thanks for your dedication and knowledge. I am looking forward to continuing working together in future projects.

Last, but of course not least. Thanks to my partner in crime and wonderful wife Catrin. To be able to succeed in life, feeling loved and motivated, you need balance. Balance at both work and home, and that's exactly what you are complete me with. Together we share sadness and joy, and together we are raising two beautiful daughters, Agnes and Alicia. For which I hope that I, as for the mentioned future young engineer, will guide, give inspiration and the support you may need on your journey.

And in the end, the knowledge (inspiration) you have is equal to the knowledge (inspiration) you make.

Victor Vestman

Örnsköldsvik, April 2023

Abstract

This thesis deals with the subject of lateral bracing between the bottom flanges of I-girder composite bridges. The focus is on the impact of adding lateral bracing on existing bridges, as well as on new bridges. Experience and knowledge from bridge projects around the world are investigated and implemented in the evaluation of the research subject.

Many existing bridges are in need of being strengthened or replaced, due to the increased traffic volume and heavier traffic loads. Different approaches can be used to prolong the lifetime of existing bridges. The approach is different depending on the cause, but for increasing the lifetime regarding fatigue some of the most suitable options are described in this thesis. A proposed concept is presented, in this thesis, along with some research questions to be answered.

The use of lateral bracings in composite bridges varies between different parts of the world. In one country it can be a requirement/common praxis for long span composite bridges with two I-girders, in other countries there are no requirements of using them. Some parts of these regulations and requirements can be traced back to the tradition in both manufacturing and construction of this type of bridges. This thesis investigates how lateral bracing is used around the world to distribute eccentric loads between primary longitudinal structural members, provide resistance to lateral loads, and to permit an existing two-girder structural system to be retrofitted to behave similarly to an often more expensive closed steel box girder.

Furthermore, several case studies have been conducted to investigate the impact on the structural behavior of composite bridges where a lateral bracing is implemented in the structure. The results from these case studies are presented in the thesis and show the advantages of the quasi-box section for which the lateral bracing is closing the composite cross section. By making the I-girder composite cross section acting more like a box-section, the distribution of eccentric loads between the girders is improved. The impact on longitudinal stresses from traffic loads and the additional effects on internal sectional parts are also evaluated and discussed.

Furthermore, proposals of the connection design for lateral bracings in existing bridges are suggested. Finally, conclusions from the results are stated.

Keywords: assessment; bridge; composite bridge; steel-concrete composite; I-girder; case study; horizontal trusses; lateral bracing; rehabilitation; strengthening; torsional stiffness; upgrading

Sammanfattning

I denna avhandling studeras effekterna av horisontella fackverk mellan de nedre flänsarna på I-balkar i samverkansbroar. Fokus ligger på de horisontella fackverkens inverkan när de implementeras i befintliga broar, men även på användandet i nya broar. Erfarenheter och kunskap från broprojekt, runt om i världen, har samlats in och utvärderas i forskningsprojektet.

På grund av ökande trafikvolymer och trafiklaster kommer ett flertal befintliga broar att behöva förstärkas i framtiden eller till och med bytas ut. För att förlänga den tekniska livslängden på dessa broar kan olika metoder användas, beroende på vilka specifika förutsättningar som råder. Några tillvägagångssätt för att förlänga befintliga broars livslängd, med avseende på utmattning, finns presenterade i detta arbete. Ett specifikt förstärkningskoncept är presenterat tillsammans med ett antal forskningsfrågor som har besvarats.

Användandet av horisontella fackverk i samverkansbroar skiljer sig åt i olika delar av världen. Det kan i ett land vara krav att använda horisontella fackverk på samverkansbroar då spannlängderna är stora, medan det i andra länder inte finns någon kravspecifikation alls gällande detta. En del av de krav som ställs kan härledas tillbaka till hur tillverkning och byggande sett ut, i olika länder, för dessa brotyper. I avhandlingen undersöks hur användandet av horisontella fackverk. för att fördela excentriska laster mellan de längsgående balkarna. ser ut runt om i världen. Det presenteras även hur dessa fackverk tillför stabilitet i sidled, samt hur ett befintligt system med två balkar kan uppföra sig likt en ofta mer kostsam stållåda.

Vidare genomförs flera fallstudier för att undersöka påverkan på det strukturella beteendet hos samverkansbroar där horisontella fackverk implementeras. Resultaten från dessa fallstudier beskrivs och fördelarna med lådverkan påvisas. Genom att få en I-balksbros sammansatta tvärsnitt att fungera mer som ett lådtvärsnitt, förbättras fördelningen av de excentriska lasterna mellan balkarna. Påverkan på de längsgående spänningarna från trafikbelastningar och ytterligare eventuella effekter på tvärförband och betongfarbana utvärderas och diskuteras.

Vidare så presenteras förslag på detaljutformning av knutpunkter i fackverk för användande i befintliga broar. Baserat på resultat och insamlad information anges slutligen slutsatser.

Nyckelord: bro; bärighet; fallstudie; förstärkning; horisontellt fackverk; I-balk; samverkansbro; samverkan stål-betong; vridstyvhet

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Notations

Roman upper letters

Symbol	Description
A	Area inside of the perimeter of a thin-walled section
A_d	Area of the diagonal member of a lateral bracing
A_F	Area of the bottom flange + ¼ of the web area
A_t	Area of the transversal member of a lateral bracing
C	Torsional stiffness
C_w	Warping stiffness
dA	An area element of a rectangular beam section
E	Modulus of elasticity of a material
EI	Flexural stiffness of a cross section
F	Stress function
F_{fl}	Force in a flange
G	Shear modulus of a material
H	Total height of a cross section
I_{1+2}	The moment of inertia for a section with two girders and a composite concrete deck
I_A	Moment of inertia for a cross section without composite action between the individual parts (wooden planks without glue at the interfaces)
I_B	Moment of inertia for a cross section with composite action between the individual parts (laminated wooden planks)
K_V	Torsion factor
$K_{v,conc}$	Torsion factor of a concrete deck
$K_{v,steel}$	Torsion factor of a steel I-girder
L	Length of a span or beam
M_{fl}	Bending moment of a flange

M_T	Torsional moment
P	General denotation for a load (point load)
Q_{m1}	Average gross weight
S	Free surface of an area element
SC	Shear centre of a cross section

Roman lower letters

Symbol	Description
b	Width of a cross section
d	Length of the diagonal member of a lateral bracing (length between conjunction points)
h	Height of an individual part of a cross section
h_c	Height of the concrete deck
h_w	Height of the web in a steel I-girder
r	Radius
s	Distance between the conjunction points of a lateral bracing
t	Thickness
t^*	Fictive thickness of a lateral bracing
t_{lfl}	Thickness of the lower flange in a steel I-girder
t_{tfl}	Thickness of the top flange in a steel I-girder
t_w	Thickness of the web in a steel I-girder
u	Displacement
w	Width
w_c	Width of the concrete deck in a steel-concrete composite bridge
w_{girder}	Distance between the steel I-girders in a steel-concrete composite bridge
w_{lfl}	Width of the lower flange in a steel I-girder
w_{tfl}	Width of the top flange in a steel I-girder

Greek letters

Symbol	Description
α	Angle between the transversal- and diagonal member of the bracing
β	Angle between the transversal- and diagonal member of the bracing
δ	Total displacement of a cross section and slip at the steel-concrete interface
δ_T	Displacement of a cross section from a torsional moment
δ_V	Displacement of a cross section from a vertical load
ε_c	Concrete strain at the steel-concrete interface
ε_s	Steel strain at the steel-concrete interface
γ_{Mf}	Partial safety factor for fatigue resistance
φ	Torsional angle
φ'	Torsional rotation
τ	Shear stress
τ_r	Perpendicular shear stress
τ_{xy}	Component of the shear stress in the xy-plane
τ_{zy}	Component of the shear stress in the zy-plane
$v_{L,x}$	Longitudinal shear force
$v_{L,x}$	Transversal shear force

Abbreviations

Abbreviation	Description
AASHTO	American Association of State Highway and Transportation Officials
BBT	Branschprogram för forskning och innovation avseende Byggnadsverk för Transportsektorn
EN	European standards – harmonized technical rules
FE	Finite Element

LCA	Life Cycle Assessment
LCC	Life Cycle Cost
LCP	Life Cycle Performance
LDF	Load Distribution Factor
LTB	Lateral Torsion Buckling
LRFD	Load and Resistance Factor Design
NYSDOT	New York State Department of Transportation
RHS	Rectangular shaped hollow sections
SBUF	Svenska Byggbranschens utvecklingsfond
WHS	Welded Headed Shear Stud

1. Introduction

1.1. Background

During the past decades European roads and railways have had an increase of traffic, by both volume and load intensity. This has led to an increased pace of new road- and railway projects to meet the demands, and bridge structures are often involved in new infrastructure projects. When a new bridge is built, one more bridge needs to be maintained. Further many existing bridges are designed to lower traffic loads and volume than today. By rather small measures a bridge service lifetime can prolonged (Habel & Harvey, 2022). The most cost-effective measure, from a socio-economic point of view, is often to strengthen the bridge to fulfill the new demands. Today, many different types of strengthening methods have been studied and in some cases tested on existing bridges.

In the Nordic countries, steel-concrete composite bridges are mainly built with a concept of a steel box-girder or two welded I-girders (Collin, Johansson & Sundquist, 2011). Both types of girder sections are used with a composite concrete deck on top. The steel box-girder concept was implemented much later than the two I-girder concept, which implies that more bridges with the I-girder section may need strengthening for increased load capacity. Since the implementing of the Eurocodes, the requirement of design regarding fatigue has been stricter. Many countries have adjusted these demands, to better consist with their previous demands, by changing the safety factor for fatigue, γ_{Mf}, or by adjusting the average gross weight of the lorries, Q_{m1} (Sousa et.al., 2019). Different approaches can be used to prolong the lifetime of existing steel- and composite bridges regarding fatigue. Some of them are:

- creating composite action for structures with no existing designed shear connection (post-composite)
- by increasing the detail category in the welded details, e.g., by post-weld treatment or reshaping the details
- by lowering the stresses by adding more material by bolting, welding, or gluing
- to change the statical system and by so lowering the cross sectional forces by distributing them between the main structural members.

This thesis is focusing on a strengthening method for the I-girder bridge cross section, where a lateral bracing is added between the bottom flanges of the I-girders. This lateral bracing has the purpose of increasing the torsional stiffness of the composite bridge section, transforming the open cross section of the I-girder system to behave more like a closed section, box-girder section. This means an approach where a combination of changing the statical system and adding more material is used to increase the resistance for fatigue and/or the load capacity (imposed load), and by so prolonging the lifetime of the bridge.

The denominations "composite bridge" and "lateral bracing" are used in the rest of the thesis and shall be regarded as synonyms for "steel-concrete composite bridge" and "horizontal trusses/bracing". In the start of this project the term "horizontal trusses" was used, but later the term "lateral bracing" was implemented, since it was found to be a more commonly used denomination in structural bridge design.

1.2. Aim and objectives

In this thesis the aim is to evaluate, understand and document the structural behavior of composite I-girder bridges with lateral bracings between the bottom flanges of the steel girders.

Two objectives are to study and evaluating the impact from lateral bracing in the design of new bridges and the impact of adding lateral bracings in existing bridges as a strengthening method. Another objective is to evaluate different approaches in the use of lateral bracings in steel girder bridges around the world.

The overall objective in this thesis is to study and verify the impact from the lateral bracings on the vertical load distribution in composite bridges.

1.3. Hypothesis and research questions

The following hypothesis was stated in the beginning of the project and has been used as a guide in the scientific work.

- Lateral bracing between the bottom flanges in steel-concrete composite I-girders bridges can be a cost-effective way to lower the steel stresses from eccentric traffic loads, both for new and existing bridges, by increasing the load distribution between the girders.

To be able to achieve the aim and objectives in this research project, the following research questions (RQ) were stated:

RQ1. To what extent has the concept with lateral bracings between the bottom flanges been used in new, and existing bridges, around the world?

RQ2. What potential benefit can the concept provide for a new composite I-girder bridge, compared to an I-girder bridge without lateral bracings or a bridge with a box cross-section?

RQ3. How much of the eccentric traffic design load can approximately be redistributed to the least loaded girder, by implementing lateral bracing on a typical Swedish composite bridge?

RQ4. How will the existing structural elements, such as cross frames and the concrete deck be affected by adding a lateral bracing?

RQ5. How could the lateral bracing be designed regarding its connection details, from design and production aspects and how could it be connected to the existing structure?

1.4. Limitations

The thesis is limited to mainly study the impact from lateral bracings on composite bridges with two steel I-girders. For other configurations with three or more girders the impact from the lateral bracing has been verified but with limited case studies. For these configurations the focus has been on the use of the lateral bracings and not only the impact.

1.5. Scientific approach

To achieve the objectives and to answer the research questions of this thesis, the following scientific approach has been used.

1) A hypothesis as a guidance for the scientific work was stated.
2) Information about the use of lateral bracings in composite bridges has been gathered. This has generally been done by performing a literature review and by contacting bridge designers and bridge owners around the globe.
3) The focusing areas for which the lateral bracing could have an impact on the bridge were decided to be; load distribution, global bending stresses and additional effects on the cross frames and the concrete deck.

4) Several case studies of composite bridges were evaluated. Different statically systems as; simple supported, continuous spans and multiple girder systems have been analyzed. The analyses have mainly focused on the impact of adding a lateral bracing between the bottom flanges of the girders.
5) The results from step 1-4 above are summarized by this thesis and in some of the additional papers presented below.
6) Design proposals of using lateral bracings in existing bridges as a strengthening technique are presented. These proposals are in the form of proposed details and recommendations for the modeling.

1.6. Outline of the thesis

The work of this licentiate thesis is presented as a compilation thesis, where a comprised summary of the appended journal- and conference papers first is presented. The core of the thesis is based on the five appended papers, and the structure of the thesis is briefly described below:

Chapter 1, Introduction: In this chapter the background, objectives and research questions, description of the scientific approach and a way of achieving them are presented. The appended papers are also introduced followed by additional publications the author has been part of.

Chapter 2, Composite bridges and their torsional stiffness: In this chapter an introduction to the concept composite action is presented together with the theory of open- and closed sections in torsion. Also, a general description about the use of lateral bracing in composite bridges is presented along with some design methods.

Chapter 3, **Existing bridges and international experiences:** This chapter contains an overview of the use of lateral bracings in some countries.

Chapter 4, Case studies: The case studies used in the appended papers are described in this chapter. The bridges are briefly described, and the purpose of the case studies are presented. Also mentioned in this chapter is earlier research in relation to some of the bridges.

Chapter 5, Impact on the structural behavior: The principles of using lateral bracing for an existing bridge as strengthening is clarified. The results from the case studies are presented and analyzed.

Chapter 6, Suggested design guidance: Suggestions for design of connections for the lateral bracing joints, for implementation in existing bridges, is presented. Some general principles and recommendations regarding modelling of the structural system are also formulated.

Chapter 7, Discussion, conclusions and suggestions for future research: The results from the research project are discussed along with some general conclusions and answers to the research questions. Furthermore, some suggestion to future research within this topic are made.

1.7. Appended papers

The essence of this thesis is built from the five appended papers listed below. The author has contributed to these papers with the general ideas behind them and writing the manuscripts. More detailed descriptions about the author's contribution can be found in the description of each paper. The five papers are written together with both researchers and designers from the own research group (supervisors) and with designers and specialist from different part of the world. The latter to be able to collect and expand the knowledge, not only from what is published by researchers but also from the experience of bridge designers and bridge owners.

PAPER I

Horizontal bracing between bottom flanges in composite I-girder bridges – A State of the Art Review

Vestman. V, Collin. P, White. H et al.

Submitted to: Practice Periodical on Structural Design and Construction, ASCE journal

This paper is a state-of-the-art review about the use of horizontal bracing between the bottom flanges in composite bridges. The concept of bracings between the flanges is investigated and the concept is exemplified by in-service bridges in Finland, Guatemala, France, and the US. The potential to use horizontal bracing as a strengthening technique in existing steel two-girder composite bridges is also discussed in the paper.

PAPER II

Box-action giving new life-time to old steel bridges

Vestman. V, Collin. P, Möller. M

Part of: Steel Bridges - 9th International Symposium on Steel Bridges, Prague 2018

The paper includes a study of an existing bridge in Sweden which was strengthened by post-installed shear connectors. Two of the spans were left to be without composite action and this paper describes the additional effects from a lateral bracing if the bridge first is strengthened by achieving composite action. The author evaluated the first assumptions and decided the limitations of the study. The author also contributed with figures and writing of the manuscript.

PAPER III

Torsion of a Norwegian bridge with partial box-action - a case study

Vestman. V, Collin. P, Oudomphanh. S

Part of: IABSE Symposium Prague 2022, Challenges for Existing and Oncoming Structures, s. 1684-1690, International Association for Bridge and Structural Engineering (IABSE), 2022

This paper describes the impact of lateral bracing on the torsional stiffness and the load distribution between the girders for an existing three span steel-concrete bridge from 1967, without composite action. The impact and importance of composite action for the concept with the bracing is exemplified and the possible effects from post installed shear connectors are also investigated. The bridge has an existing lateral bracing, and its impact and limits are investigated and compared to different variations of the bracing.

PAPER IV

Strengthening of a Composite I-girder Bridge by Trusses Introducing Box-Action

Vestman. V, Collin. P, Hällmark. R

Part of: ECCS 10th International symposium on steel bridges – for A Green Planet, Istanbul 2022.

This paper includes numerical investigations regarding the use of post-installed bracing systems between the lower flanges in twin I-girder composite bridges. In this study the author considered additional shapes of the trusses, compared to earlier studies by the author. Also, the impact on existing structural parts as shear studs, support- and internal cross-frames are investigated. The Yxlö Bridge in Sweden was chosen as a case study in this paper.

PAPER V

Lateral trusses between I-girders introducing torsional stiffness to a composite bridge in Guatemala

Vestman. V, Garcia. J, Caballero. G, Collin. P

Submitted to: Eurosteel 2023 Amsterdam, the 10th Eurosteel Conference.

This paper describes the design of a curved bridge in Guatemala City and its challenges. The new Bridge over the Pinula River is designed as a steel-concrete composite bridge with multiple steel girders with a concrete deck on top and has lateral bracing between the girders. The paper investigates the impact of lateral bracings, and especially bracings between the lower flanges, for traffic loads in the serviceability limit state (SLS) and for fatigue traffic loads. The author has decided the limitations of the study and has evaluated the results for the different traffic loads. The model and the output of the results were done by the second author, who was responsible for the design and had a prepared model that could be modified in this study.

1.8. Additional publication

In addition to the five appended papers the author has published the following papers in the field of bridge design. Some of them within the scope of this thesis, others not. These papers are not appended in this thesis but are listed below. These papers have given the author the opportunity to collaborate with other researchers and engineers in the field of bridge engineering. Also, the author has got the possibility, and was given the experience to present the work for different audiences by presenting in different formats as: papers, (science- and popular science) articles, and technical reports. The contribution from the author in each publication is described below.

CONFERENCE PAPERS:

Improvement of Fatigue Resistance through Box-Action for I-Girder Composite Bridges

Vestman. V, et. al

Part of: IABSE Congress, Stockholm, 2016: Challenges in Design and Construction of an Innovative and Sustainable Built Environment, 2016, pp. 1988–1994

This paper covered a case study regarding the impact on the remaining lifetime for fatigue of a continuous composite bridge in Sweden designed according to the European design codes, EN. The results from different trusses (shapes and stiffnesses) between the bottom flanges of the bridge were compared and analyzed. The author of the thesis modelled the bridge in a FE-program and analyzed the results. The author also did the main part of the writing and layout of the manuscript.

Additional effects from transforming open bridge cross section to semi-closed

Ivanov. S, Collin. P, Vestman. V

Part of: IABSE Symposium Wroclaw 2020 – Synergy of Culture and Civil Engineering

This paper covers the impact of lateral bracing for the case study in the paper, "*Box-action giving new life-time to old steel bridges*". The author contributed with general assumptions and limitations for the study. Most of the work regarding modelling and presentation of the results were made by the first author of the paper. The author of the thesis also contributed with the analysis and conclusions regarding the results for the additional effect on the concrete deck.

Slussen – The Lock of Stockholm

Vestman. V, Svensson D
Part of: IABSE Congress Ghent 2021 - Structural Engineering for Future Societal Needs

This paper covers the main structures of the new Slussen in Stockholm. The old structures, both the bridge and the canal and the foundations of those were passing their service life. The paper described the background behind the structures and their design. The author was responsible for the description and explanations regarding the steel bridge, which he has been part of designing. Both the design and construction method were described along with the special assembling method which included sea transportation and barges.

Monitoring of a Norwegian steel-concrete bridge strengthening for composite action

Vestman. V, Collin. P, Hällmark. R, Arason. M
Part of: IABSE Congress Ghent 2021 - Structural Engineering for Future Societal Needs

This paper is a conference version of a coming journal paper describing the results and findings from the monitoring of an existing bridge before and after strengthening with composite action. The bridge was strengthened with coiled spring pins acting as shear connectors at the steel-concrete interface and the same bridge was monitored before and after the installation of the shear connectors. The author was responsible for the modeling of the bridge, analysis of the results and comparison with the measurements from the monitoring. Also, the author was responsible for the writing of the manuscript. The second author of the paper was responsible for the preparation of the measurement procedure and the decision of which parameters that was monitored.

Testing of composite girders with coiled spring pin shear connectors

Hällmark. R, Nilforoush. R, Vestman. V, Collin. P
Part of: IABSE Congress Ghent 2021 - Structural Engineering for Future Societal Needs

This paper describes tests of two composite girders before and after installation of shear connectors (coiled spring pins) at the steel-concrete interface. The two girders were monitored before and after the installation and stresses, deflections, and slip (steel-concrete) were measured. The author of the thesis assisted with the preparations before the tests and also with some verifications by a FE-model regarding the behavior of the girders.

TECHNICAL REPORT:

Prolonging Lifetime of Old Steel and Steel Concrete Bridges, final report

Collin. P, Hällmark. R, Vestman. V et al.
(RFSR-CT-2015-00025). ISBN-13: 978-92-76-17327-4

This is the final report of the European RFCS-project ProLife. The project concerned several methods of strengthening existing bridges to prolong their lifetime. The author of the thesis contributed to two work packages with both writing of the manuscript and conference papers included in the project.

Förlängning av gamla stålbroars livslängd genom lådverkan, final report
Vestman. V, Collin. P
Final report to SBUF in 2019, (SBUF ID: 13287)

This is the final report to Svenska Byggbranschens utvecklingsfond (SBUF), where parts of the research project presented in this thesis were summarized. The author of the thesis contributed with the underlaying research which had been presented in several conference papers. The author also contributed with figures and conclusions regarding the proposed design method for adding lateral bracing in existing I-girder composite bridges.

POPULAR SCIENCE ARTICLES:

ProLife – förstärkning av befintliga stål- och samverkansbroar [in Swedish]
Vestman. V
Bygg & Teknik, nr. 2 2016, s 61–62

This is an article describing, at the time, the work in the RFCS-project Prolife. The author was responsible for the information and the writing of the manuscript.

Innovasjon i bruforsterkning i Agder, beskrivning av förstärkning för samverkan samt mätningar på norsk bro, före och efter förstärkning [in Norwegian]
Arason. M, Gundersen. E, Collin. P, Vestman. V
Nyheter om Stålbyggnad nr 4 2020, Stålbyggnadsinstitutet.

This is an article about the strengthening work on two Norwegian bridges where Coiled Spring Pins were used as shear connectors. The article includes both the design work for the strengthening and the monitoring of one of the bridges. The author was responsible for some illustrations regarding the results from the comparison between the analysis of the bridge and the monitored measurements.

Slussenbron, en avancerad stålkonstruktion med ett speciellt montage [in Swedish]
Bäck L., Svensson D. & Vestman V
Nyheter om Stålbyggnad, SBI, nr 1 2020, s 36–39.

This article is a popular science version of the conference paper "*Slussen – The Lock of Stockholm*"**.** The author of this thesis was responsible for the illustrations and the writing about the steel bridge.

WORKSHOP PROCEDING:
Sustainable steel-composite bridges in built environment
Collin. P, Vestman. V, Nilsson. M
Ramboll Sverige AB & Luleå University of Technology, Luleå, March 2018.

This is a report of the workshop which was a part of the dissemination project SBRI+ (RFCS 2016-710068). The workshop was held to disseminate the project "Sustainable steel-composite bridges in built environment and its results. In the research project a holistic approach was applied to steel-composite bridges by combining analyses of Lifecycle Assessment (LCA), Lifecycle Costs (LCC) and Lifecycle Performance (LCP). The author was responsible for collecting the material from the workshop combined with some of the papers written in the project. These materials were combined in a report which was published and printed.

2. Composite bridges and their torsional stiffness

A common bridge type in Sweden is the steel-concrete composite bridge. This type of bridge consists of steel girders with an overlaying concrete deck. To achieve an economical and a material optimized construction the girders and the deck are connected with shear connectors at the steel-concrete interface (Collings, 2005). The traditional way of achieving composite action is by using welded headed shear studs, WHS, at the steel-concrete interface. This principle of using composite action between the two cross section parts is often used when designing new steel-concrete bridges according to the European norms, Eurocode. Existing road bridges in Sweden designed before the mid 80´s, were often not designed with any intentional composite action. Nowadays, composite action is required in steel-concrete composite bridges that are continuous over supports, in line with the Swedish bridge code (TRVINFRA-00227, 2022).

The tradition in design of these steel-concrete bridges varies around the world, but in principle the design is similar. About the history on the evolution of steel-concrete composite construction a brief review is presented by Pelke & Kurrer (2015), Hicks et al. (2016) and Lam (1998). The superstructure consists of the steel girders, the concrete deck and additional cross section parts as the cross bracings. These are the main parts for the traditional steel-concrete girder bridge. In addition, horizontal bracing is used between the top and/or the bottom flanges. The bracings can have different purposes as stabilization in the construction stages or stabilization against horizontal loads, such as wind loads. To stabilize the cross-section system for wind loads, the bracing is often called wind-bracings.

2.1. Composite action

The composite action of structures means that parts, instead of acting as individual members, the parts are connected and acts as a single structural member. The composite action can be between parts of the same material or with different materials. For bridges the term composite action is often referred as composite action of parts of different materials. Two examples of composite structural members are laminated timber, where the glue acting as shear connection between the wooden planks, and reinforced concrete. The last is however usually not referred as a member with composite action, even if that term is correct by the definition of composite action. In steel-concrete composite bridges, the composite action is referring to the connection between a steel- and a concrete member. For steel-concrete bridges the use of composite action is an efficient way of using the two materials, utilizing the tensile strength of the steel and the high compressive strength of the concrete (Hanswille, 2011). The composite action of the steel-concrete cross section is achieved by shear connectors, which can be designed in different shapes (Liu, Bradford & Ataei, 2017). The shear connectors need to be able to resist the separation forces, which wants to separate the two materials. For this, the shear connectors are designed to have sufficient strength, stiffness and ductility to enable a single structural member of the steel and concrete parts. A way of designing theses structural members is described in the European codes (EN 1994-2, 2005).

For a simply supported steel-concrete girder the difference between the behavior of a single structural member (composite), and where the two structural parts acting individually (no composite), is illustrated in Figure 1. The illustration shows at the left a girder without shear connectors (non-composite steel-concrete girder), where a slip (δ) will occur at the steel-concrete interface when the girder is loaded. This implies that plane sections will not remain plane during bending. The two parts, steel and concrete, will share the applied load in proportion to their individual flexural stiffness (EI). To the right the opposite case, where it is no slip at the steel-concrete interface due to the shear connectors (composite steel-concrete girder, is illustrated. In the latter case it can be assumed that the plane section remains plane during bending, according to the Euler-Bernoulli beam theory.

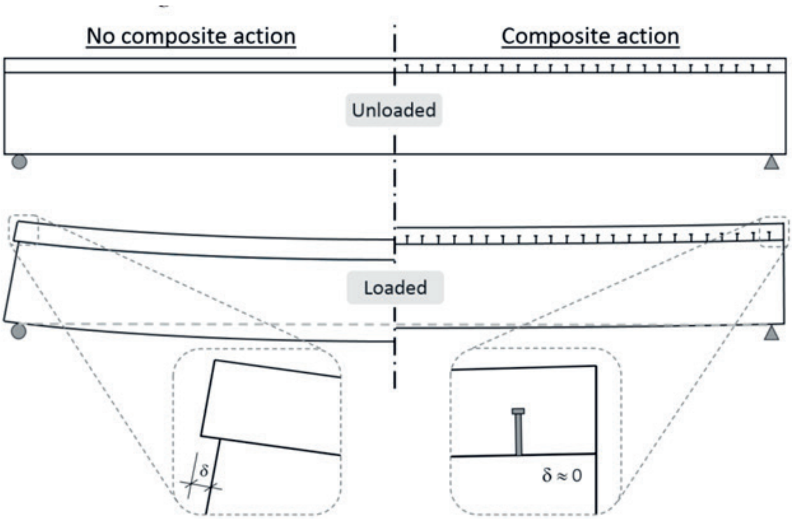

Figure 1 *No composite action vs. composite action (Hällmark. R, 2018).*

Friction and other interlocking factors are disregarded for the design in EN 1994-2. The main idea with using the composite action between the two sections is to increase the stiffness of the whole cross section. This can easily be shown by comparing the equations (Eq. 1 and Eq. 2) for the moment of inertia for the two different types of cross sections, for a laminated timber beam, see Figure 2. Where the stiffness of the full-composite section (I_B) is governed by the fact that the total height (H) is used instead of the individual height (h) of each plank as for the non-composite section (I_A) in (Eq. 3).

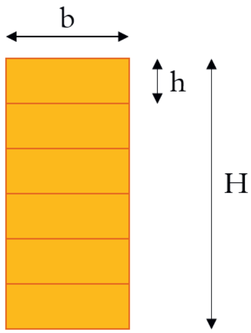

Figure 2 *Cross-section of a laminated timber beam.*

$$I_A = \sum \frac{bh_i^3}{12} \qquad \textit{Eq. 1}$$

$$I_B = \frac{bH^3}{12} \qquad Eq.\ 2$$

$$I_B > I_A,\ since\ \ H^3 \gg \sum h_i^3 \qquad Eq.\ 3$$

This can further be described by the change in the structural response regarding strains in the section. By looking at the cross section in Figure 3, consisting of a steel I-girder and a concrete deck on top, the different strain distributions are illustrated for the two cases, no composite- and composite action. The defined slip (δ) is a consequence of the differences between the steel strains, ε_s, and the concrete strains, ε_c, at the steel-concrete interface. For the case with composite action, it is assumed to be such stiffness of the shear connector that no slip will occur at the interface. Contrary, for the case with no composite action, no interlocking forces are assumed to be present. Therefore, the slip in this case will only be dependent on the individual stiffness for the two parts.

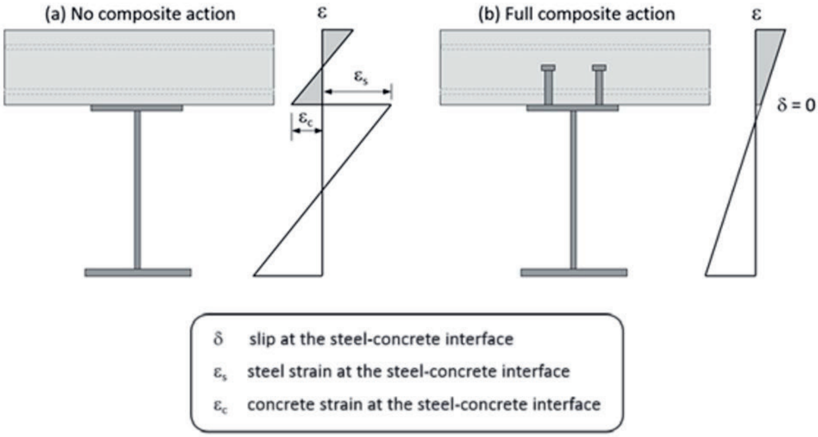

Figure 3 *Illustration of the vertical strain distribution for cross sections with (a) no composite action and (b) full composite action (Hällmark. R, 2018).*

These two cases illustrated above are extremes, where the real behavior is something in between (Hällmark, R 2018). This behavior is denoted as partial-composite action but will not be explained further in this thesis.

2.2. Closed section vs. open sections

For an I-girder bridge with two girders, which also is straight and symmetrical, the dead load can be assumed to be distributed evenly between the girders. An eccentric load, may it be superimposed dead loads or traffic loads, will be unevenly distributed. The distribution will be in such way that the girder closest to the load will carry a larger proportion of the load. However, a closed section like a box-girder will distribute the load more evenly between the two main girders, where the load is equally shared for the ideal case. The difference in load distribution between the two sections, I-girder and box-girder, is simplified and illustrated in Figure 4. For this to be the case, the box-girder section needs to have sufficient internal intermediate cross frames or diaphragms to prevent distortion of the cross-section which would lead to out of plane bending stresses in the webs and full-width bottom flange.

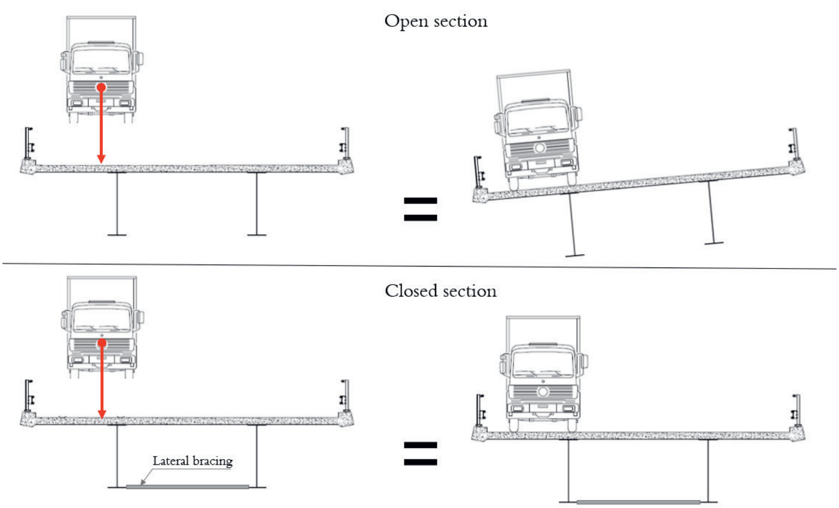

Figure 4 *Displacement/rotation from an eccentric load for a I-girder composite section with- and without lateral bracing.*

In Nylander. H (1973) the theory of torsion for beams with open- and closed cross sections is explained. Parts of this theory, which is of importance to understand the essence of this thesis, is summarized below.

Consider a beam with a constant cross section as in Figure 5 on which a torsional moment (M_T) is applied in each end. The torsional moment is constant along the beam length (L).

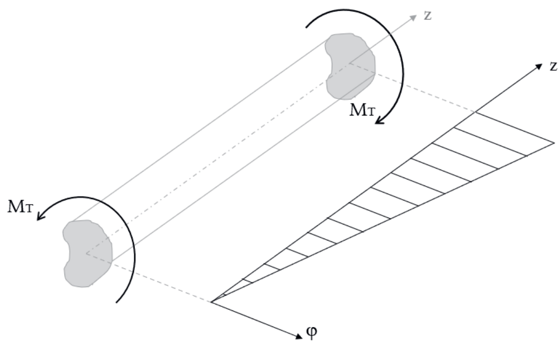

Figure 5 *Illustration of a beam with a constant cross-section loaded by a torsional moment (After: Nylander, 1973).*

If the cross section of the beam at both ends can be assumed to be unchanged by the applied torsional moment, the change of the torsional angle (φ') is the same along the whole beam. This can be written as

$$\frac{d\varphi}{dz} = \varphi' = constant \qquad Eq.\ 4$$

For elastic materials and where the deformation is small, the correlation between change of the torsional angle (φ´) and M_T can be derived as,

$$\varphi' = \frac{M_T}{C} \qquad Eq.\ 5$$

where C is a constant that denotes the torsional stiffness of the beam, which can be described with the following expression:

$$C = GK_V \qquad Eq.\ 6$$

where

G – is the shear modulus of the material

K_V – is the torsion constant for the cross section.

With the expressions in Eq. 5 and Eq. 6 the following expression for the torsion constant can be derived:

$$K_V = \frac{M_T}{G\varphi'} \qquad Eq.\ 7$$

The difference in the torsional angle of a cross-section that is torsional plane and a twisted cross-section is illustrated in Figure 6. Where A is the distribution of the torsional angle for the non-warping cross section (the cross section remains plane during torsion), and B is for the warping cross section. The beam is fixed for twisting at both ends.

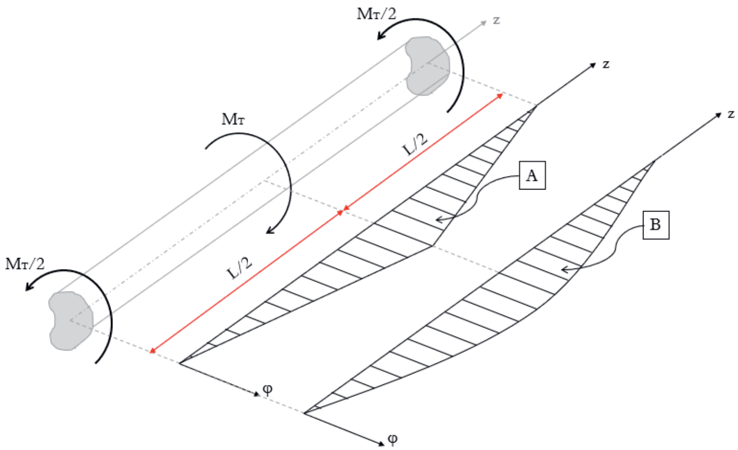

Figure 6 *Torsional angle (A-non warping cross section, B-warping cross section) along a beam loaded in the middle by a torsional moment (After: Nylander, 1973).*

Cross sections that are circular or hollow circular will remain plane during an applied torsion. For a rectangular cross section this is not applicable. This limitation can be described by showing the assumption that sections remain plane conduct that shear stresses on an element (dA) are acting perpendicular to the direction of the radius vector (r). Figure 7 shows this perpendicular shear stress (τ_r), for the element dA in a rectangular beam loaded by a torsional moment. Parallel to the x- and y- axis the shear stress is divided into the components, τ_{xy} and τ_{zy}. According to the theory of elasticity it should be equal shear stresses τ_{xz} and τ_{yz} in perpendicular areas against the element dA. This mean that on the free surface (S) τ_{yz} would act, which is impossible because it is not loaded. Therefor it is not applicable to use the theory, which is based on that sections remain plane, for stress calculations on rectangular beams. It can only be applied on circular or hollow circular sections.

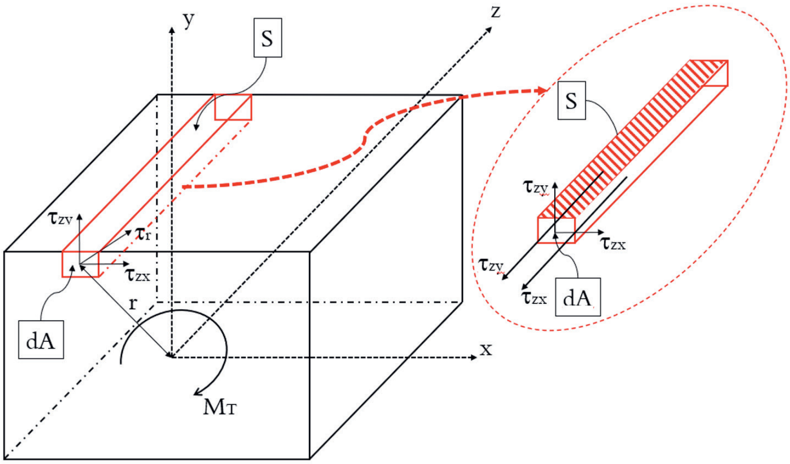

Figure 7 *Principal of the plane section theory for a rectangular beam section (After: Nylander, 1973).*

If the derivation for the theory of Saint-Venant´s is neglected the basic equation could directly be expressed as Eq. 8 and further derived by implementing it on the section in Figure 8.

$$\frac{\delta^2 F}{\delta x^2} + \frac{\delta^2 F}{\delta y^2} = -2G\varphi', \ which\ is\ constant \qquad Eq.\ 8$$

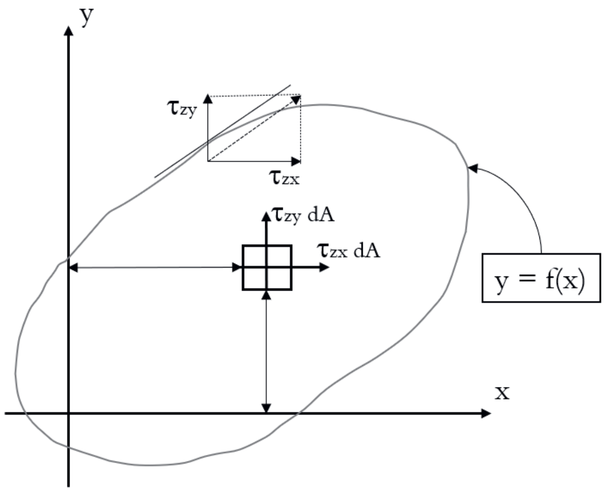

Figure 8 *Section of a beam loaded with a torsional moment (After: Nylander, 1973).*

The stress function (F) can be defined by the two following equations:

$$\frac{\delta F}{\delta y} = \tau_{zx} \qquad Eq.\ 9$$

$$\frac{\delta F}{\delta x} = -\tau_{zy} \qquad \textit{Eq. 10}$$

The boundary condition for which the shear stress should be parallel to the boundary limitation curve, see at the top of Figure 8, could be expressed as Eq. 11 on the curve y = f(x).

$$\frac{dy}{dx} = \frac{\tau_{zy}}{\tau_{zx}} \qquad \textit{Eq. 11}$$

By using the condition in Eq. 11 combined with Eq. 9 and Eq. 10 the statement that the stress function, F is constant on the curve y = f(x). The torsional moment for massive cross sections could be derived as in Eq. 12 if the constant is set to zero.

$$M_T = \iint \left(\tau_{zy}x - \tau_{zx}y\right)dxdy = 2\iint F\,dxdy \qquad \textit{Eq. 12}$$

The calculation of the torsion factor for different type of cross section can be found in the literature, e.g., in Nylander. H (1973). The torsion factor for a closed arbitrary thin-walled shaped section and an open section which are of interest for this research project is illustrated below with the derived equations.

For an arbitrary closed thin wall section as in Figure 9, the torsion factor can be calculated as

$$K_V = \frac{4A^2}{\int \frac{ds}{t}} \qquad \textit{Eq. 13}$$

Where A is the area inside the dashed line in Figure 9.

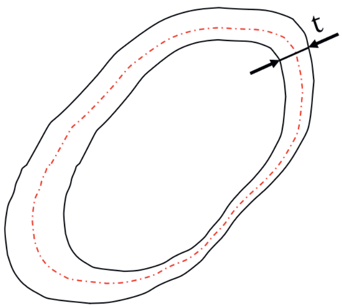

Figure 9 *An arbitrary thin-walled closed cross section.*

For an open cross section, the same factor can be simplified with following equation:

$$K_v = \sum_{i=1}^{n} \frac{h_i t_i^3}{3} \qquad \textit{Eq. 14}$$

where

h – is the length of the different parts in the section

t – is the thickness of the part in the section

This equation varies for different shapes of open sections. The torsion factor should be multiplied by a factor c, which is typically between 1,0 (e.g., for a L-shape) and 1,3 (e.g., for an INP-section).

It can be noted that the torsion factor for a closed section is much greater than for an open section.

The difference between an open and closed section can also be illustrated by comparing the shear flow in the two sections. Both the shear flow and the internal lever arm are completely different between the two types, as can be seen in Figure 10.

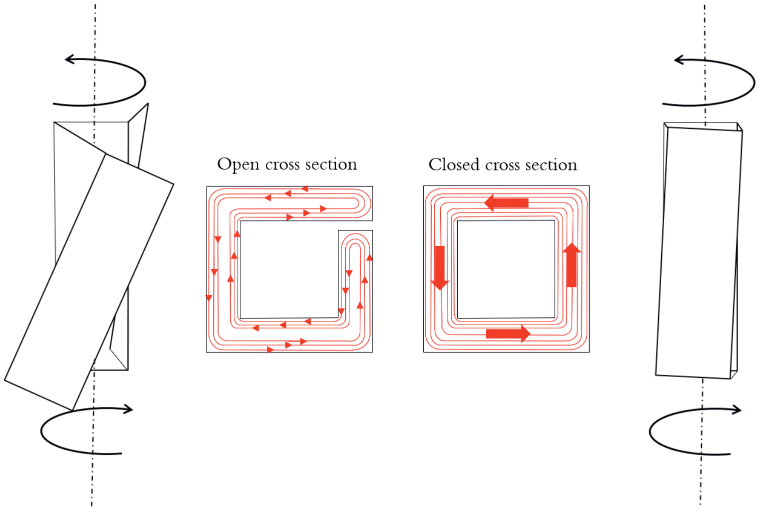

Figure 10 *Comparison of the shear flow for two different cross sections.*

For a cross section (doubly symmetric) with warping stiffness the lateral torsion can be illustrated as in Figure 11.

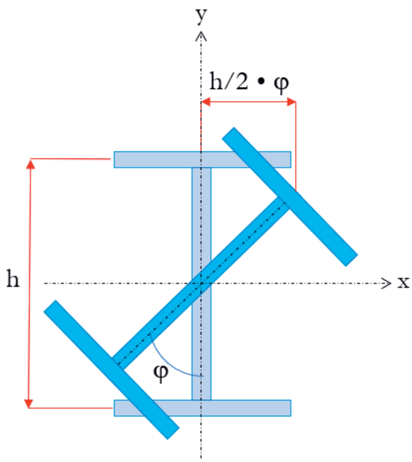

Figure 11 *Lateral torsion of a double symmetrical cross section (After: Nylander , 1973).*

For the beam the displacement (u) in the transversal direction (x) can be derived as

$$u = \frac{h}{2} \cdot \varphi \qquad \textit{Eq. 15}$$

The bending of the flange (M_{fl}) in the same direction is

$$-M_{fl} = EI_{fl}u'' \qquad \textit{Eq. 16}$$

By insert Eq. 15 in Eq. 16 the expression for the bending moment will be

$$-M_{fl} = EI_{fl}\left(\frac{h}{2}\cdot\varphi\right)'' \qquad \textit{Eq. 17}$$

By deriving the Eq. 17 an expression for the forces in the flanges (F_{fl}) is

$$EI_{fl}\cdot\frac{h}{2}\cdot\varphi''' = \frac{dM_{fl}}{dz} = -F_{fl} \qquad \textit{Eq. 18}$$

The lever arm for the force in the flanges is the distance between the centroid of the flanges, in Figure 11 noted as h. The expression for the torsional moment (M_T) in the beam will then look like

$$M_T = F_{fl}\cdot h = -EI_{fl}\cdot\frac{h^2}{4}\cdot\varphi''' \qquad \textit{Eq. 19}$$

A more general equation of Eq. 19 is

$$M_T = C_w\cdot\varphi''' \qquad \textit{Eq. 20}$$

where

C_w – is the warping stiffness of the cross section.

The expression regarding both torsion- and warping stiffness of the cross section will therefore be

$$M_T = C\cdot\varphi' - C_w\cdot\varphi''' \qquad \textit{Eq. 21}$$

2.3. Increased torsional stiffness of open cross sections

One solution to transform the behavior of an open section to the behavior of a closed section, also called quasi-closed section, is to add a lateral bracing between the bottom flanges of the girders. The bracing can be arranged in different shapes, where the most common shapes are with single diagonals (D-shape), K- and X-shape (Fan & Helwig, 1999). A more detailed description of lateral bracing in bridges and possible shapes of the bracing configuration is found in Ch. 2.3.1.

With respect to torsion, the lateral bracing between the bottom flanges, makes the flanges and bracing act as a homogeneous flange, as in a steel box-girder. The increase of the torsional stiffness can be illustrated by the difference between the torsional stiffness of an open and a closed cross-section. Figure 12 illustrates how the increase of the torsional stiffness can be described in the change of the shear flow in the cross section. Closed cross sections can carry the torsion from eccentric loads by shear flow around the entire cross-section. The shear stresses (τ) shown in Figure 12 are caused by the St. Venant component of the applied torsional moment (M_T).

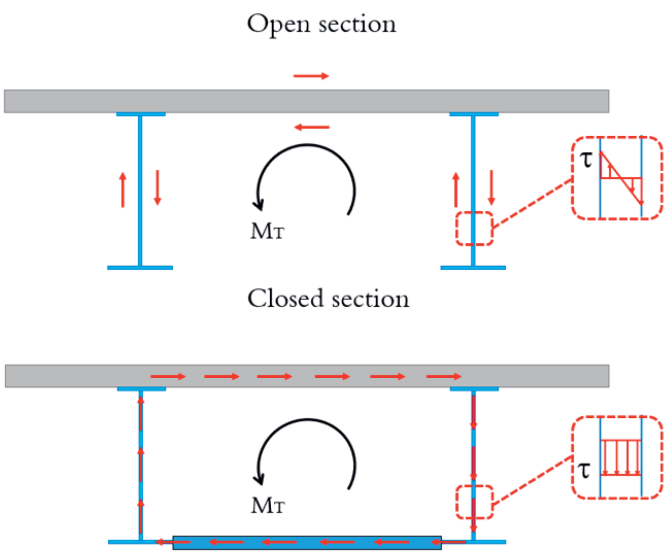

Figure 12 Shear flow from St. Venant torsion in an open- and closed cross section.

2.3.1. Lateral bracing

For composite bridges a lateral bracing can be used as stabilization for torsional moments and horizontal loads. These stabilization systems are often needed during the construction stages for bridges with open sections as, trapezoidal-, and I-girder sections. The trapezoidal section is often the cross section of a composite box-girder during construction (Fan & Helwig, 1999). For comparison a composite box-girder in service can typically have a torsional stiffness around 100 to 1000 times higher than a comparable composite I-girder section. The large torsional stiffness makes the box-girders attractive for application in horizontally curved bridges, in which the bridge geometry may result in large torsional moments on the cross section. Even if the final composite box-girder bridge has a large torsional stiffness, the cross section during transportation and construction has a relatively low stiffness due to its open shape. Due to that the cross section at these stages can be subjected to large torsional moments, an introduction of some sort stabilization system is relevant. Additional internal cross frames or diaphragms could be used to increase the torsional stiffness (Fan & Helwig, 1999). Even if this method is possible, it is not often used. These cross frames or diaphragms are mainly used to control the distortion of the cross section from eccentric loads. Another possible solution which was stated earlier, is to fasten a lateral bracing between the top flanges, see Figure 13.

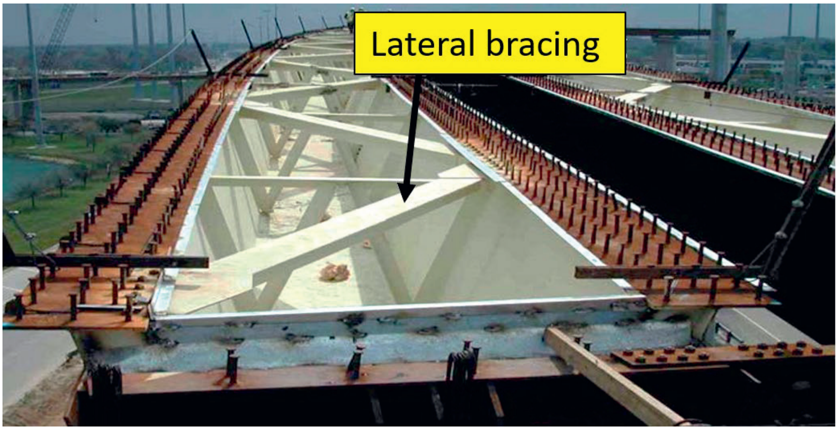

Figure 13. *Lateral bracing between the top flanges of a trapezoidal box-girder during construction (Helwig et.al., 2007).*

This method is commonly used for open section during construction but could maybe be beneficial for open sections in service, like a composite I-girder bridge. Different types of shapes for the lateral bracing between the top flanges of a trapezoidal girder section was evaluated by Rageh (2012). The shapes included in the study were X-, Warren, Pratt and K-shape. These configurations have also been found to be the most used shapes (Fan & Helwig, 1999), where X- or K-shaped bracing often is the preferred configuration (Berthellemy, 2002). In Figure 14 the four different shapes for the lateral bracing are shown, together with the denotations used further in this thesis.

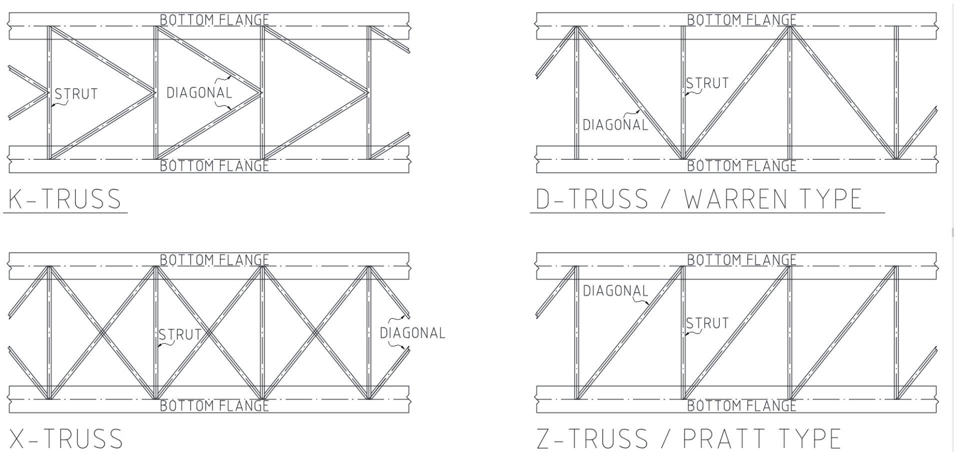

Figure 14 *Shapes of the lateral bracing between bottom flanges (Paper I / Vestman et.al., 2023).*

2.3.2. Design methods

To be able to calculate the torsional stiffness of the cross section with the additional lateral bracing, by hand, a model presented by (Kolbrunner & Basler, 1969) can be used. Using this model, the bracing can be approximated as a fictive thickness (t^*). To be able to calculate the closed section of the I-girder composite section with the additional lateral bracing as a closed box-section, a transformation of the lateral bracing into an equivalent continuous sheet needs to be done. The analysis provides that the shear flow in the closed section is transformed to forces in the bracing members. By comparing the shear stiffness of the sheet with the corresponding shear stiffness of the bracing members using the virtual work principle, the fictive thickness of the bracing can be determined.

Depending on the cross section areas of the bracing, the geometrical properties of the girders and also the geometry of the bridge, the approximated thickness can be calculated for different bracing constellations. Below, the K-shaped bracing is illustrated and it's equation derived according to (Roik, 1983). According to (Lorentsen et. al, 1979) should at least ¼ of the web area be included in the area denoted as A_F.

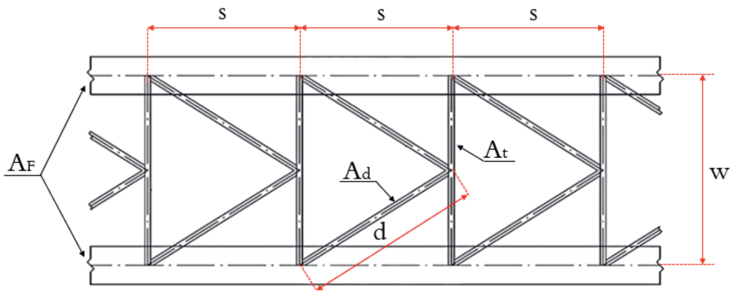

Figure 15 *K-shaped bracing and properties for the fictive thickness calculation.*

$$t^* = \frac{s \cdot w}{\frac{2d^3}{A_d} + \frac{w}{4A_v} + \frac{s^3}{24A_F}} \qquad \textit{Eq. 22}$$

where

s – is the distance between the vertical members of the bracing between the girders

w – is the center distance between the girders

d – is the length of the diagonal member of the bracing (length between conjunction points)

A_d – is the area of the diagonal member of the bracing

A_t – is the area of the transversal member of the bracing

A_F – is the area of the lower flange of the girder including a part of the web, (Lorentsen et. al, 1979).

To validate and to visualize the concept of lateral bracing between the bottom flanges with the theory of fictive thickness, a concept bridge was presented in a master thesis (Vestman, 2016). The bridge was both modelled in a FE-program, SOFiSTiK, and calculated with analytical methods. Both methods confirmed that the theory described earlier in Ch. 2.2.1, combined with the theory described in this chapter gives equivalent results of the load distribution between the girders. Below the calculation example from (Vestman, 2016) is derived and further explained.

The bridge used in this example is denoted as the "concept bridge". The concept bridge is a simple supported composite bridge. The superstructure consists of two welded steel I-girders, which have constant cross sections, along the bridge and a composite concrete deck with constant width and thickness. The dimension of the superstructure is described and illustrated in Table 1 and Figure 16.

Table 1 *Properties of the concept bridge.*

General	
Length (L)	30 m
Concrete deck	
Width (w_c)	11 m
Height (h_c)	370 mm
Steel girders	
Distance girders (w_{girder})	6000 mm
Thickness top flange (t_{tfl})	20 mm
Width top flange (w_{tfl})	400 mm
Height of web (h_w)	2533 mm
Thickness of web (t_w)	19 mm
Thickness of lower flange (t_{lfl})	25 mm
Width lower flange (w_{lfl})	800 mm

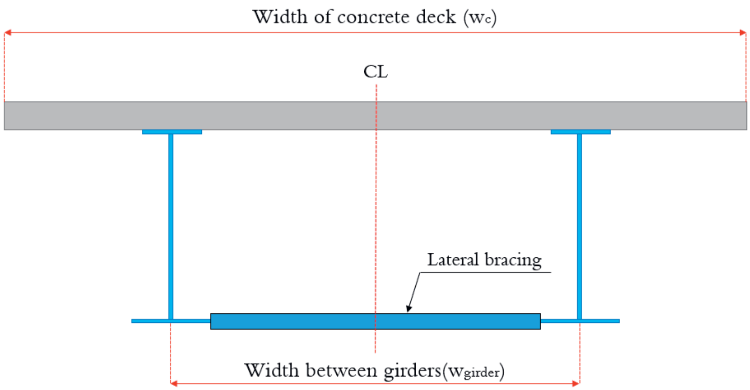

Figure 16 *Cross section of the concept bridge.*

The bridge was modelled with both beam- and shell elements. The preferred way of modelling in the program SOFiSTiK is with a so-called composite section. By using this type of combined element, the design forces could directly be used in the software for verification or together with other post-calculation verifications. In (Vestman, 2016) and (SOFiSTiK AG, 2014) the method of modelling is described in detail. The boundary conditions are modelled as nodal supports at the bottom of the steel girders. The support conditions for the bridge are as for a determinate system where the bearing configuration is with one fixed-, one unidirectional- and the other two as multidirectional bearings.

The effective thickness depending on the angle between the diagonals and the bridge is illustrated in Figure 17. The calculations indicates that a suitable angle of the diagonals in a K-truss could be somewhere between 35–50 degrees for this specific bridge and bracing member. Rectangular shaped hollow sections (RHS) with a profile 250x150x10 mm are used for the bracing members.

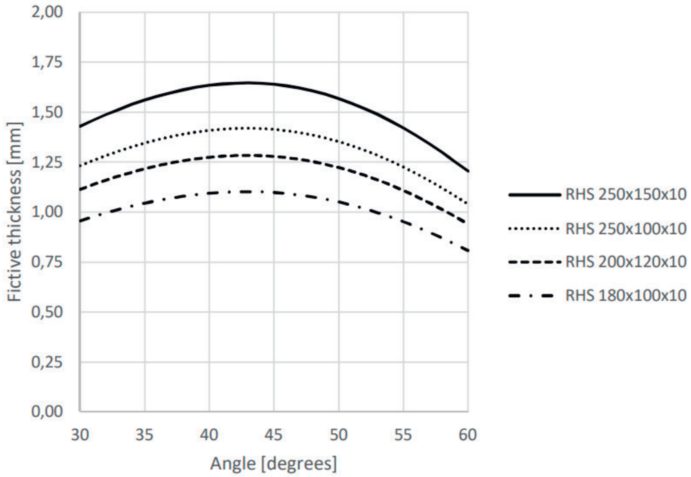

Figure 17 *Fictive thickness according to the theory by (Kolbrunner & Basler, 1969) and (Roik, 1983) for a concept bridge (Vestman, 2016).*

For the analysis of the impact from the lateral bracing, a K-shaped bracing with a chosen angle of 45 degrees between the diagonal and the transversals was chosen. In Figure 18 the modelled bridge with the K-shaped bracing is shown.

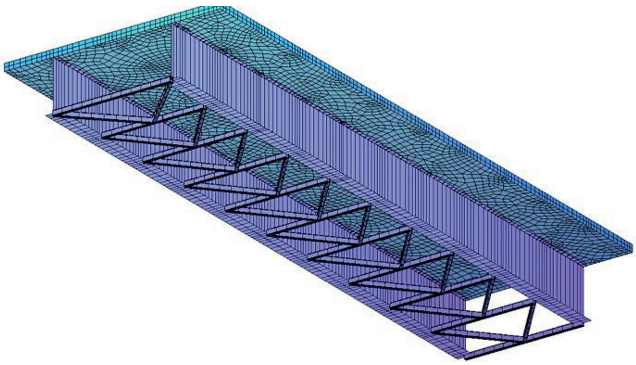

Figure 18 *Concept bridge from below showing some of the structural elements.*

The load case studied in this verification analysis is a simple point load (P). The point load is located at the middle of one of the girders, in the middle of the bridge span (L). The magnitude if the of load is 1 MN.

By using the theories describes in Ch. 2.2.1 the deflection of the bridge girders can analytical be derived. The first thing is to describe the deflection from the eccentric load in two components, vertical- and torsional deflection, see Figure 19. The vertical component is calculated as the deflection of a simple supported beam loaded with a point load in the middle. Because the point load is assumed to be taken by both girders the load is half of the applied load, which is the same as calculating the deflection for the total stiffness of both girders. The parameters in Eq. 23 are defined in Figure 19 and Table 2

$$\delta_V = \frac{PL^3}{48EI_{1+2}} \qquad \textit{Eq. 23}$$

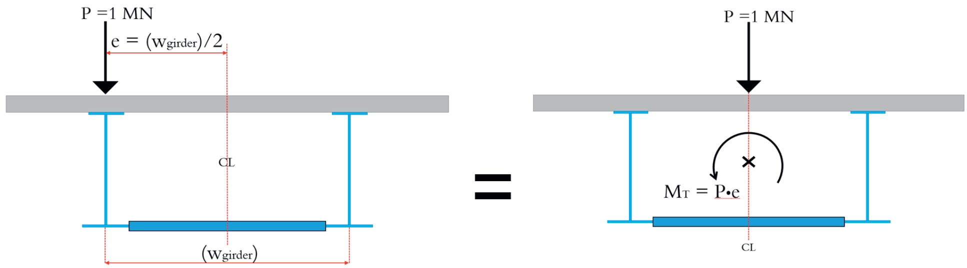

Figure 19 *Equivalent torsional moment and centric point load to an eccentric point load.*

The displacement from component of the torsional moment (M_T) can be derived from the rotation of the cross section.

$$\varphi = \frac{L}{2}\varphi' \qquad \textit{Eq. 24}$$

$$\varphi' = \frac{M_T}{C + C_w \frac{\pi^2}{L^2}} \qquad \textit{Eq. 25}$$

Where the additional factor π^2/L^2 comes from the differential solution of Eq. 20. From the torsional angle (φ) the displacement (δ_T) depends on the length between the supports as:

$$\delta_T = \sin(\varphi)\frac{w_{girder}}{2} \qquad \textit{Eq. 26}$$

The total displacement (δ) at the middle of the loaded girder will therefore be the sum of the two components as:

$$\delta = \delta_V + \delta_T \qquad \textit{Eq. 27}$$

The cross-sectional properties are derived and presented in Table 2. The resulting displacement in Eq. 27 is then compared to the calculated value from the FE-model in Vestman. V, (2016). From the theory described in Ch. 2.3.2 and the curve of RHS 250x150x10 mm in Figure 17 a value of the fictive thickness (t^*) is given as 1,6 mm.

Table 2 *Cross-sectional properties of the bridge composite section*

Properties	
Fictive thickness (t^*)	1,6 mm
Moment of inertia (I_{1+2})	$4{,}96 \star 10^{11}$ mm^4
Torsion factor steel section ($K_{v,steel}$)	$1{,}82 \star 10^{12}$ mm^4
Torsion factor concrete deck ($K_{v,conc.}$)	$2{,}76 \star 10^{11}$ mm^4
[1)]Torsional stiffness (C)	$4{,}83 \star 10^{16}$ Nmm^2
[2)]Warping stiffness (C_w)	$9{,}40 \star 10^{23}$ Nmm^4

[1)]Is the combined torsional stiffness of the composite section (steel girder, bracing and concrete deck).

[2)]Is the combined warping stiffness of the composite section (steel girder, bracing and concrete deck)

From the values in Table 1 and Table 2 the torsional angle in Eq. 24 is calculated to $9{,}0 \star 10^{-4}$ rad. The total displacement at the middle of the bridge is then calculated by Eq. 27

$$\delta = \delta_V + \delta_T = 5{,}41\ mm + 2{,}62\ mm = 8{,}03\ mm$$

Table 3 *Comparison between the analytical and the FE-calculated displacement.*

Model	**Displacement**
Analytical 2-D	8,03 mm
[3)]FEM 3-D	8,89 mm

[3)]Is taken from the results in (Vestman, 2016).

Further this displacement from the analytical analysis can be compared to the displacement of the bridge without a lateral bracing. By excluding any effects from warping, this can be done by implementing the same theory as previously, meaning that the eccentric load will be taken as a vertical displacement, but now only by one girder because the lack of torsional stiffness of the cross section. This displacement would be twice the displacement in Eq.23 and therefor 10,82 mm. This is an increase of 20–35 % from the calculate displacement in Table 3. This means that the lateral bracing for the concept bridge reduces the displacement and the longitudinal bending stresses by 18–26 % for the applied load case.

3. Existing bridges and international experiences

The regulations and requirements regarding the use of lateral bracings in composite bridges varies between different parts of the world. Some countries specifically require the use of lateral bracing (Liikenneviraston, 2016) while others have no limit or requirements of their use. Some of the reasons for this disparity can be traced back to the type of structures that the country typically constructs. For example, in the Nordic countries, composite bridges are often made with only two welded steel I-girders. These are very efficient for spanning over rivers with steep embankments, where it is often no problem with the free height under the bridge. These girders might benefit from a horizontal bracing during construction and while in service for stability problems. In contrast, bridges in other countries like Luxembourg are often constructed with a bridge cross-section consisting of multiple rolled steel beams (Lorenc & Kożuch, 2016). These types of bridges are often more stable for stability issues as lateral torsional buckling, LTB, and do not benefit from the horizontal bracing until the span lengths become quite long or have a small horizontal curvature radius.

In Paper I, a collaboration between bridge designers in different countries was established. Following sections are a summary of that work, where experiences from different parts of the world are described together with some examples of bridges with lateral bracing. The bridges are briefly described below, and a more complete information can be found in the paper.

3.1. Finland

Lateral bracing for the purpose of distributing vertical loads has been used in Finland for large steel beam bridges since the late 1970s (Lilja. H, personal communication, October 7, 2021). Lighter steel beam profiles with an open cross section, whose main purpose is stiffening against horizontal loads during erection and service, were already in use well before the 1970s (Lilja. H, personal communication, October 7, 2021).

The main functions of lateral bracings, usually constructed from hollow rectangular steel sections (RHS), are to stiffen the structure against horizontal loadings and to add torsional stiffness to the structure. In the first bridges constructed with horizontal bracings, they were mainly used to enhance torsional stiffness (Lilja. H, personal communication, October 7, 2021).

The behaviour of a composite I-girder cross section with lateral bracing between the bottom flanges is almost analogous to that of a box-section. National guidelines in Finland advise that the use of horizontal bracing is advantageous for span lengths exceeding 50–70 m (Liikenneviraston, 2016). In practice, this advice is often taken as a requirement and any bridge span exceeding 70 m automatically uses horizontal bracing (Lilja. H, personal communication, October 7, 2021).

Typically, a K-type bracing between I-girders is used in Finland. The K-type is advantageous since it doesn't behave as a bottom flange in the main girders. On the other hand, it also doesn't induce additional stresses to the main girders, if the system nodes coincide.

Similarly, the vertical transversal bracing is of a single K-type, opening upwards. It is customary that the top chord of transversal truss is temporary (Nilsson, 2012). If the top chord is not removed, it must be designed against concrete slab shrinkage. Where the distance between the main girders is large, a single K-type truss may not be possible and one or more pairs of K-type diagonals are added, see Figure 20 over the "*Mälkiä channel bridge A*", built 2010. In such case, it is not possible to remove the top chord. Normally a horizontal gusset plate (penetrating the bottom chord) is used to connect horizontal diagonals, see Figure 21.

Figure 20 *Pair of vertical K-bracing. Source: Heikki Lilja*

Figure 21 *Connection between lateral- and vertical (cross) bracing. Source: Heikki Lilja*

Two examples of bridges with lateral bracings are the Tervola bridge, built in 1975 with a main span of 72 m and the Kaitainen bridge, Figure 22 built in 1982 with a main span of 90 m Currently, Finland has about 35 steel girder bridges with horizontal bracing.

Figure 22. *Kaitainen bridge. Source: Heikki Lilja*

Another example of a Finnish bridge is the Jännevirta bridge, Figure 23, built in 2018. It is one of Finland's top ten longest bridges. The bridge is almost 600 m long and has a deck width of 15 m. The longest span is 120 m, and the superstructure is built with steel I-girders that haunch down at the intermediate supports. Both the Kaitainen bridge and the Jännevirta bridge utilized a K-shaped truss for the horizontal bracing system.

Figure 23. *Jännevirta bridge. Source: Ramboll*

3.2. France

Twin I-girders with bracings are commonly used in France for bridges carrying two tracks for high-speed railways. The increased torsional stiffness associated with the use of the bottom lateral bracing is beneficial to limit the girder deflections due to eccentricity when a train crosses the bridge. The reduced deformation translates into reduced vertical acceleration and the ability to satisfy the comfort rules can be ensured. A more expensive approach to reduce the deflection would be to increase the stiffness of each individual girder.

An example where lateral bracings between two I-girders has been used instead of a box-girder solution, is the Lille Porte Sud Bridge in France, see Figure 24. At the time of the design and construction of this bridge the general approach for curved bridges in France was to use box-girder sections (Berthellemy, Labourie & Leconte, 2002). For a concrete bridge this might often be true due to the advantages of a cross section with a uniform torsional stiffness (Savio & Prasada, 2017). For that reason, the bridge was first designed with a steel box-girder, but this solution was too expensive (Berthellemy et.al., 2002). In connection to this, an alternative solution was made. This solution consisted of a design with two I-girders and to reduce the effects from fatigue loads in the regulations, Eurocode, a lateral bracing was used. The purpose of the lateral bracing was to increase the load distribution from eccentric traffic loads by using the benefits by a more torsional stiff cross section. This design saved around 15 % of the total steel weight compared to the box-girder solution (Berthellemy et.al., 2002).

Figure 24 *Street view from below on the Lille Porte Sud Bridge, (Google, n.d.).*

The Porte Sud Bridge was completed in 2001 and the final designed consisted of two main girders with a constant height of 2,2m and spaced at 7 m between each other. Between the bottom flanges a lateral bracing was used, as described above. During the construction stage, a temporary bracing between at the top part of the girders was needed, see Figure 25. This due to the strong horizontal curvature and since the cross section consisting of the I-girders and the lateral bracings between the bottom flanges is like a U-section. For this U-cross section, the center of torsion and the center of gravity are located in quite different positions, making the cross-section sensitive for torsion. Due to the condition during the construction, the temporary lateral bracing at the top of the section was almost a mandatory solution.

Figure 25. *Bottom lateral bracing between the bottom flanges and temporary bracing at the top part of the girders. Source: Jacques Berthellemy*

3.3. United States and South America

In the United States (US) and most Latin American countries, the main bridge design code is the AASHTO LRFD Bridge Design Specifications (LRFD). This code is used in some specific types of bridges, and it provides simplified rules valid only for a limited range of geometries and structural configurations.

LRFD deals with two types of composite bridges, I-girder and box girder steel and concrete composite bridges. I-girder bridges are usually composed of more than two girders. The reason is that in a bridge with only two main girders, the longitudinal girders are considered fracture critical members while this does not apply for bridges with three or more girders. Fracture critical members require more fabrication quality control and design checks than non-fracture critical girders. A typical example of a bridge structure with multiple girders is the bridge that carries Route 17 WB over the Chenango River near Binghamton, NY. The bridge is a fully Integral Abutment Bridge using curved weathering steel girders under construction in the NY City area. The concrete deck is placed on foam filled stay-in-place forms and the bottom lateral bracing is used only in the exterior bays, see Figure 26. This bridge is an eight-span curved girder bridge composed of weathering steel.

Figure 26. *Bridge from underneath visualizing the multiple girders, cross bracing and lateral bracing at the outer bays. Source: Harry White*

Initiated by the author and Harry White at the NYSDOT, a survey was sent to all 50 Departments of Transportation in the US regarding the requirements that each state had concerning lateral bracing. Of the 30 agencies that responded, only 3 agencies had provisions that required the use of lateral bracing. For straight girders, 2 agencies encouraged the use of horizontal bracing for spans greater than 43 m (149 ft.) but would permit their exclusion if calculations showed that they were not necessary. For curved girders, 1 agency required use of lateral bracing regardless of the degree of curvature. It should then be no surprise that the standard details from the Federal Highway Administration do not show the presumptive use of lateral bracing but provides useful rules and recommendations. In fact, the most widely used software programs for the design and rating of steel bridges (ex. MDX and LEAP), do not consider the structural contribution of lower horizontal bracing in their analysis.

One bridge example from the South America, where lateral bracing is used, is the Oxec Bridge in Guatemala. The bridge has a longitudinal inclination of 1% and is composed of three tangent spans with lengths of 49,5 m, 77 m, and 49,5 m for an overall length of 176 m. The structure was designed and checked to be incrementally launched from one end. The superstructure consists of a composite concrete deck, with a total deck width of 9,6 m, supported by three continuous 3,2 m high steel I-girders with a spacing of 3,4 m.

Figure 27 *The Oxec Bridge in Guatemala. Source: PEDELTA*

4. Case studies

In four of the five appended papers (Paper II-V), existing bridges have been used as case studies to evaluate the behavior and impact of adding lateral bracing between the main steel girders. In Paper I, existing bridges have also been used. Then to clarify the use of lateral bracing in existing composite bridges as a State of the Art, rather than case studies for analytical evaluations. In the following section the bridges are briefly described with an explanation of how they were used as case studies in this research project. Some of the bridges have been used in other research projects, which also is mentioned in the descriptions below. Together these four case studies capture bridges, simply supported and continuous, without existing lateral bracing between the lower flanges and bridges, twin- and multiple girders, with existing lateral bracing.

4.1. Paper II – Pitsund Bridge

In Paper II the Pitsund Bridge in northern Sweden was used to analyze the behavior of a two-span composite bridge if a lateral bracing is added. The Pitsund Bridge is a seven-span bridge, with a total length of 399 m and a free width of 9 m. In six of seven spans, the superstructure consists of steel girders with a concrete deck slab on top. The seventh span is a movable span, designed as a bascular bridge with two leafs.

The bridge has three different types of cross-section compositions. The first part which was strengthened in 2006 with coiled spring pins (Hällmark, 2018) to achieve composite action is partly shown in Figure 28. The second part which was original designed as a composite cross section with welded shear headed studs in 1984. The last two spans consist of a non-composite section with steel I-girders and an overlaying concrete deck, with only a few anchoring rebars that connect the steel and the concrete. This part of the bridge was chosen for the case study. The idea was that if a strengthening should be needed for this part of the bridge, the first part of the strengthening should be with the same method used in the first part of the bridge, with the coiled spring pins (Hällmark, 2018). If needed the bridge could be strengthened further with a lateral bracing. This part of the bridge was monitored by Hällmark & Collin (2019) to establish the eventual composite action between the concrete deck and the steel girders. The load distribution between the two girders was also monitored for different load positions. For the most eccentric load position the LDF was measured to around 0,96 (6,50 respectively 0,30 mm deflection) at the middle of the span (Hällmark & Collin, 2019).

Figure 28. *The Pitsund Bridge from below at the southern end.*

For the case study in this research project a fatigue load model was used in the analysis. This to establish the impact on the remaining lifetime of the bridge regarding fatigue. The bridge was modelled in ANSYS with shell- and beam elements, where the concrete slab is modelled by 8-node solid elements. The steel girders are modelled by 4-node shell elements and the cross-frames and lateral bracing are modelled by beam elements. Three different shapes for the lateral bracing were analyzed, see Figure 29. In the analysis the stiffness of the bracing members was also varied.

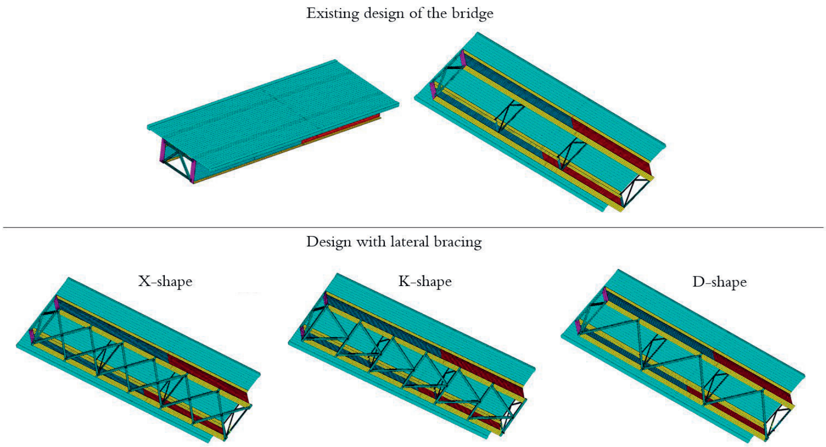

Figure 29. *Segments of the modelled Pitsund Bridge, existing- and strengthened design.*

4.2. Paper III - Østre Trøsken Bridge

The strength of the Østre Trøsken Bridge in Norway had been evaluated regarding traffic load capacity and found to be limited by LTB of the flanges from bending moments. The bridge was built in 1967. The construction is a welded steel girder bridge with two identical girders in three spans and a concrete deck consisting partially of prefabricated elements. The total length of the bridge is 96m with the longest span being 51m for the center span, see Figure 30.

Figure 30. *The Østre Trøsken Bridge, overview of the bridge from north (Paper II / Vestman et.al., 2022).*

The bridge has an existing lateral bracing, consisted of relatively slender cross sections. These bracings were probably designed to stabilize the bottom flanges against horizontal loads, as wind. In Paper II the bridge was analyzed regarding the impact from the existing lateral bracing but also the impact of composite action between the concrete deck and the steel I-girders was evaluated. The bridge was evaluated for five different configurations, which were modelled in SOFiSTiK with shell element (concrete and I-girders), beam elements (bracing and cross frames/beams). For the five configurations the shear connection at the steel-concrete interface was varied and the bridge was also evaluated with and without the existing lateral bracing. The change of the composite action was included in the analysis to verify its importance for the lateral bracing to have an impact on the structural behaviour. No additional lateral bracing was implemented in this study, only the existing bracing was analysed and evaluated.

4.3. Paper IV – Yxlö Bridge

The Yxlö Bridge in Sweden is a simply supported bridge in one span of 31 meters over the Yxlö channel south of Stockholm, see Figure 31. The bridge was built in 1961 and consist of two steel I-girders with a concrete deck on top of the girders. The bridge was design with no intentional composite action, without shear connectors. The impact of different degrees of composite action was studied by Tjernberg (2021).

Figure 31 *The Yxlö Bridge in summertime. Source: The Swedish Transport Administration, n.d.*

For the study in Paper IV, the cross section was considered with full composite action, meaning that the shear connection at the steel-concrete interface was regarded as rigid. Seven different configurations of the bracing system were evaluated for their impact on the load distribution between the girders, the impact on the existing cross frames and the impact on the longitudinal shear flow at the steel-concrete interface. The different configuration of the lateral bracing is shown in Figure 32. In the analysis the Fatigue load model 4 from EN 1991-2 was used.

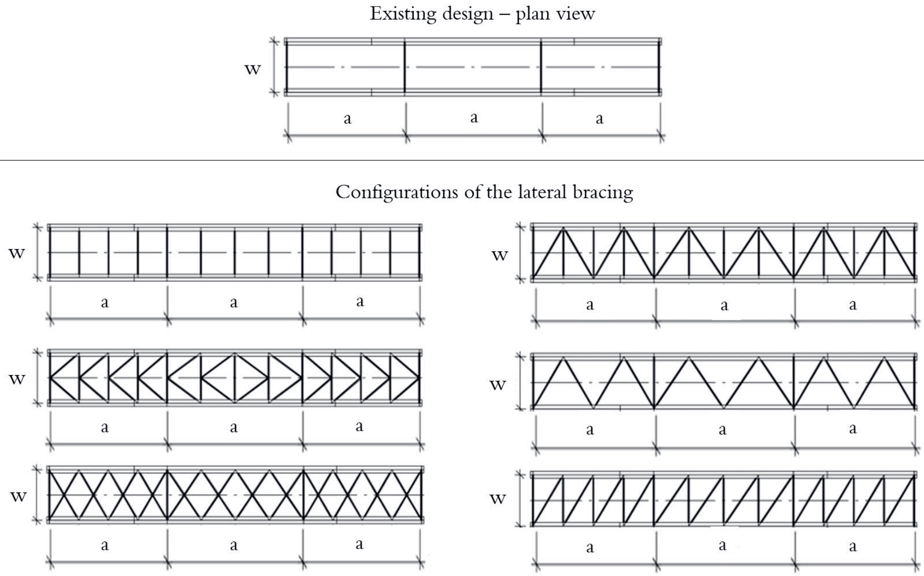

Figure 32 *Existing bridge design (plan-view) and the additional six configurations where a lateral bracing is added.*

4.4. Paper V - Bridge over the Pinula River

The new Bridge over the Pinula River is part of the new south access road to Guatemala City called VAS ("Vía Alterna del Sur", which means "Alternative Southern Carriageway"). The bridge is designed as a steel-concrete composite bridge with welded steel I-girders with a overlaying concrete deck. It has an overall length of 161 m, divided into three uneven spans, with lengths of 51+50+60 m. The bridge is curved in plan, with a curvature radius of 148 m and a constant longitudinal slope. The cross-section of the bridge with the six longitudinal I-girders is shown in Figure 33.

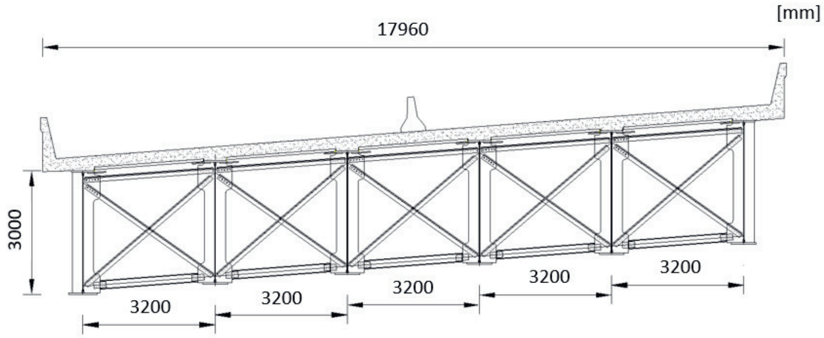

Figure 33 *Cross section of the Pinula Bridge (Paper V /Vestmanet.al., 2023).*

The bridge was designed without lower bracing at most of its length, bracings were only placed at the support regions as indicated in Figure 34. This configuration was established to provide flexibility during the launching stages, which would not have been the case with lateral bracings continuously over the bridge length. A torsional stiff cross section together with the different precamber of the I-girder, would have made that only one or two out of the six girders should have been in contact over the temporary bearings during launching. Nevertheless, in Paper V a comparison of the effects in service conditions from adding lateral bracing between the bottom flanges and with the actual design was made. For the analysis the same type of load, HL-93 live load model, as in the design was used. The model for the analysis was made in SAP-2000, as the design for the existing bridge.

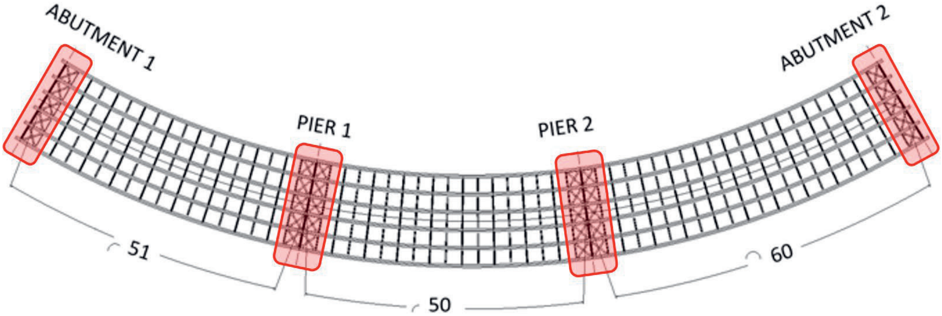

Figure 34. *Plan-overview of the distribution of the lateral bracing between the lower flanges for the existing bridge. (Paper V / Vestman et.al., 2023)*

5. Impact on the structural behavior

5.1. Load distribution

For a composite bridge with two main longitudinal girders, the load effect from eccentric live loads can be calculated with the assumption that the concrete deck is simply supported between the two girders. Using a simple statics analysis, this results in a load effect factor greater than 1.0 when the load is placed on the cantilevers of the concrete deck. Even using a more precise method, such as a 3D finite element (FE) analysis, the load distribution factor (LDF) will be shown to be around 1,0. Since the torsional stiffness of the concrete deck and the warping stiffness of the whole composite section will transfer some of the load and even out the distribution. For the reviewed case studies, the load effect factor for the most eccentric load case is approximately 1,2 when a simply supported deck is assumed and 0.95 when a rigorous 3D FE analysis is used.

When a horizontal bracing system is added to a twin steel I-girder bridge with a composite concrete deck, the increased torsional stiffness reduces the load distribution factor from 1,0 to approximately 0,7, as illustrated in Figure 35.

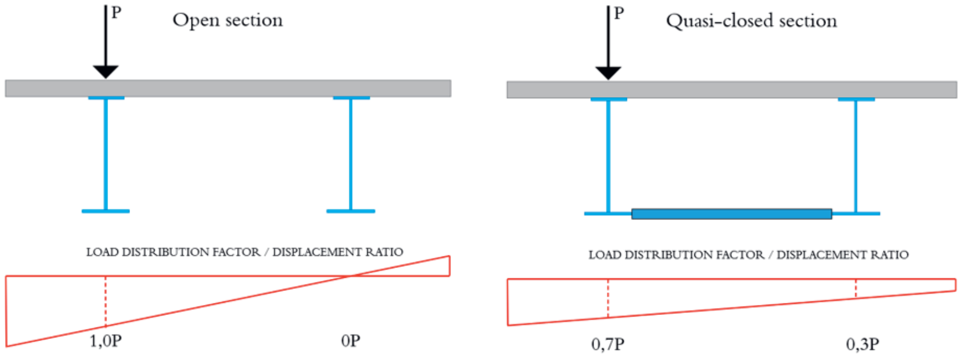

Figure 35 *Load distribution factor or displacement ratio of an eccentric unit load (P) for an open- and a quasi-closed composite bridge section.*

5.2. Steel girders

In Paper II the impact on the stresses in the bottom flanges were studied for different shapes of the lateral bracing. The same was studied in and Paper III with an additional analysis regarding the impact of the lateral bracing combined with the necessary composite action between the concrete deck and the steel girders.

In Paper II the focus was mainly on the impact on the fatigue stresses, where a fatigue load model according to EN 1994-2 was used in the studied load case. The stress amplitude was studied for the bridge without any lateral bracing and with three different shapes of the bracing (D-, K- and X-shape). The impact from the stiffness of the bracing members was also analyzed. Two different profiles were used for the lateral bracing. For bridge used in the study the stress was reduced by 20–40 % for the different configurations. This reduced stress amplitude, or increased load distribution by the lateral bracing would increase the remaining theoretically lifetime of the bridge by 2,5–12 times regarding fatigue load and the fatigue detail in the web stiffeners.

A Norwegian bridge in need of strengthening was studied in Paper III. The bridge had an existing lateral bracing, and this specific bracing was used for the study, which implies that no variation of the shapes or stiffness was implemented in the analysis. One conclusion which was stated is that some bridges are not designed to resist torsional moments. The bracing in this case were denoted as wind bracings on the original design drawings and were therefor most likely not designed to take the normal forces caused by the torsional moment and the quasi-closed section. The behavior of this kind of bridge will nowadays be captured by any FE-modelling, where such effects as additional normal forces from torsion would be managed.

5.3. Cross frames

In Paper IV the impact from lateral bracing between the bottom flanges on the internal cross frames was studied for a simply supported composite I-girder bridge in Sweden. The bridge had been studied earlier in (Tjernberg, 2022) regarding a post-installation of shear connectors, were different levels of composite action at the steel-concrete interface were analyzed. In Paper IV the analysis showed that the normal forces increased in the cross-frame members when a lateral bracing was added to the structure. Different shapes of the lateral bracing were analyzed, and the conclusion was that shape of the bracing had a small influence on the result regarding increased normal force. However, one of the shapes was an outlier regarding this, and that was a bracing were only additional transversals were used. The transversals were designed like the bottom member of a cross frame and was probably too weak and widely spaced to be able to distribute the torsional moment and have an impact on the statical system of the bridge.

5.4. Concrete deck and steel-concrete interface

In (Ivanov, Collin & Vestman, 2020) the impact on the concrete deck from the lateral bracing was analyzed, for a simply supported I-girder composite bridge. Below the conclusions from that study is discussed.

When closing an open bridge cross section with additional steel bracing, the cross section starts to resist applied torsional moments by uniform torsion. This means that following changes in the stress state of the concrete deck might be expected (Ivanov et. al., 2020):

- Reduction of the normal force in the deck as well as reduction of the longitudinal shear flow from vertical shear due to the more evenly distributed vertical load between the main steel girders
- Reduction of the torsional moment taken by the concrete deck itself due to the torsional stiffness of the closed cross section
- Increase in the longitudinal shear force from uniform torsion due to the participation of the middle part of the concrete deck (between the steel girders) into the quasi-box, see Figure 36 for an applied torsional moment around the shear centre (SC).

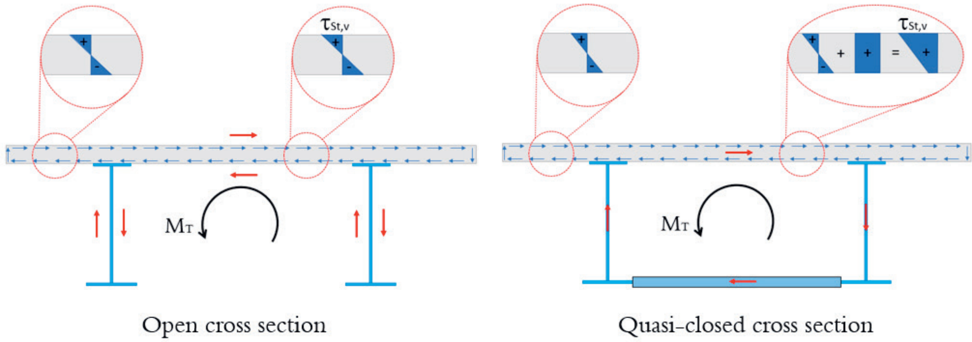

Figure 36. *Shear stress in the concrete deck from (uniform) torsional moment, (After: Ivanov et.al. 2020).*

The following conclusions was derived from the study:

Due to the higher St. Venant's torsional stiffness of the quasi-closed cross section, the torsional moment taken by the concrete deck is reduced with around 50% compared to the open cross section (Ivanov et. al., 2020).

Due to the better load distribution of the eccentric loading between the main girders in the quasi-closed cross section, the maximum normal force per meter in the concrete deck over the loaded girder is reduced.

In the quasi-closed cross section, the shear force in the plane of the deck, due to vertical shear in the steel girders, reduces at the loaded girder and increases at the other one. This because of the better distribution of the vertical load between the two girders. On the other hand, the uniform torsion taken by the quasi-closed section leads to an increase of the shear force in the deck, between the main steel girders. This increase is estimated to be of around 20% (Ivanov et al., 2020). This effect must be considered in longitudinal shear verification for the concrete deck since for eccentric load the longitudinal shear in the concrete deck is not the same at both sides of the loaded girder for the quasi-closed cross section.

The maximum longitudinal shear flow $v_{L,x}$, in the shear connection of the loaded girder, is reduced by 7% for the quasi-closed cross section (Ivanov et. al., 2020). The positive reduction, from the better load distribution between the two girders, combined with the negative increase of the shear flow due to the torsion taken by the quasi-closed cross section, leads to this reduction.

In the case of the quasi-closed cross section the vertical bracings resist the distortion of the cross section by maintaining its shape and introduce the applied torque. This leads to additional loading of the shear connection in y-direction (perpendicular to the steel girders axis) into the cross sections with vertical bracings. The magnitude of this local force in the connection is calculated to approximately 25% of the longitudinal shear force (Ivanov et. al., 2020). This effect deteriorates in cross sections without cross bracings.

In Paper IV similar conclusions, as for the study (Ivanov et. al., 2020), were made regarding the distribution of the shear flow at the steel-concrete interface. Not mentioned in Paper IV is that the shear force component in the transversal direction increases around the area of the internal cross frames. The cross frames control the distortion of the cross section from torsional moments if they are adequately spaced along the bridge. The shear flow will however be increased locally due to the restraint of the out-of-plane bending stresses of the flanges (of a quasi-closed section) by these intermediate cross frames.

6. Suggested design guidance

Based on the experiences both from the international examples (Ch.5) and the case studies carried out by the author (Ch.4), some advice is given by the author in this chapter.

6.1. Arrangement of the bracing members

In general, the effectiveness of the new lateral bracing (K- or X-bracing) is dependent on the existing cross beams. It was found in (Collin et. al., 2015) that both the stiffness and the geometry of cross beams have a substantial influence in the effect, especially in cases where the existing cross beams are non-symmetric profiles such as low UNP-profiles. It has also been showed in Paper II that for the three types of lateral bracing (K-, X- and D-shape) all are quite equally efficient for the purpose of distribute the eccentric load between both girders.

The two types of trusses shown in Figure 37 can both take torsion from eccentric loading. One big difference between the two types is that the X-shape also will take part in the global bending of the superstructure, while the K-shape, when loaded by global bending, will escape by deformations of the members perpendicular to the bridge. To be able to increase the bending stiffness of the composite cross section by the X-shaped bracing, transversal members are needed at the joints between diagonals and bottom flanges, otherwise the bottom flange will deform perpendicularly to the longitudinal axis of the bridge, introducing some lateral bending stresses. These stresses will of course be included in the results of a FE-analysis of the bridge modelled with shell elements. In many cases the cross girders can be close and rigid enough to take care of this issue.

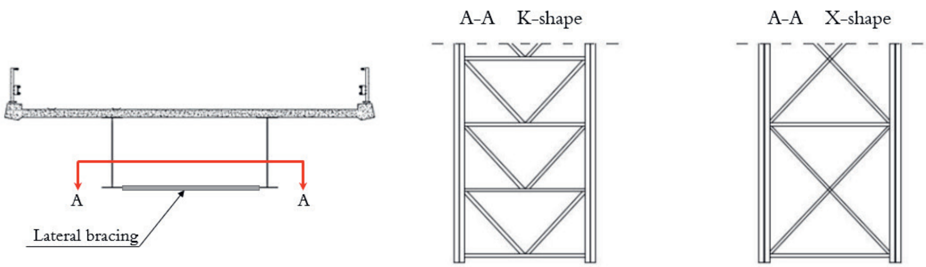

Figure 37 *Two types of shapes for the lateral bracing (K- and X-shape).*

Without any additional members at the locations for the connection of the bracing the global stiffness contribution is limited, even for X-bracings that would have given a decreased stress in total for the global bending. However, with an additional member between the flanges, or the case with the position of the diagonal-flange connections coinciding with the original cross girders and web stiffeners, the horizontal truss will carry a part of the global bending moment for the case of X-shape or D-shape. For the K-shape the lateral bending of the member perpendicular to the bridge flanges means that this effect will be negligible.

The angles between the diagonals are another parameter that will be influenced by the distance between the girders, as well as the longitudinal distance between the joints of the truss. If the cross girders are close and the distance between the girders is quite large, it might be enough to have one X-shaped bracing per cross girder, meaning that no extra members perpendicular to the bridge are needed in between the cross girders, as the cross girders will keep the distance between the bottom flanges at each joint. The effective thickness depending on the angle between the diagonals and the bridge is briefly described in Ch. 2.3.2. For the specific bridge the indication is that a suitable angle of the diagonals in a K-truss could be somewhere between 35–50 degrees.

6.2. Modelling of the structure

A first recommendation is to apply a FE-model based on shell elements for the steel girders to ensure that the relevant effects are included and thereby that the effects of the new bracing members not are overestimated. Special attention should be applied if beams models are applied – especially in cases where the existing cross beams and web stiffeners are sparse, hence not able to prevent some distortion of the box-section that is created when a truss is fastened to the bottom flanges. If beams models are used for analysis, it is recommended that these models should be verified and/or calibrated to avoid results that might be unsafe. It is important to include original cross-bracings and web stiffeners in the model since they prevent the distortion of the cross-section.

As for the bridge deck it may be modeled by either shell elements or solid elements. Solid elements increase the model size but give a slightly better representation of the structure because the geometry of the deck may be accounted for precisely. The increase in model size is not dramatic and so the bridge deck can be modeled by solid elements. For bridges with welded shear connectors between deck and girders, perfect bond may be assumed.

As for cross-bracings in bridges, and horizontal trusses between bottom flanges, these may be modeled by beam elements for the purpose of establishing the static behavior of the bridge. If the connections of the beams to the bridge girders are such that it may be considered hinged, end release of considered rotational degrees of freedom is conducted. It should be noted, that for the static behavior of the bridge, the impact of the connections being hinged or rigid on the behavior of the bridge is small. However, for the resistance of the beam element it matters.

When it comes to fatigue, the cracking of the concrete above the internal support has little influence on the results for a two-span continuous bridge. The increased field moment because of the decreased support moment is almost taken out by the decreased field moment from when the load is on the other span. The 15% rule of clause 5.4.2.3 in EN 1994-2 is however recommended, together with modelling only half the thickness of the deck in shear (Vayas & Iliopoulos, 2013).

The local effects near the new joint connections should be analyzed. The study in this research project has not revealed any substantial stress concentration, but it is recommended that the local stresses are studied in the joint connection, adjacent web stiffeners, gusset plates, bottom flange of main girder etc. Such analyses are partly dependent on the exact layout of the joint connections. If considered critical, such analysis could be carried out in a FE-model using solid elements to be able to model the behavior correctly including local eccentricities, gusset plates, bolts, etc.

An interesting observation is that the sum of both deformations and stresses of the two girders in an I-girder bridge are the same whether the truck is placed eccentrically or in the middle bridge. Furthermore, both the stresses and the deformations of the loaded girder decrease quite simultaneously when the bridge is modelled with a lateral bracing between the bottom flanges. This is however not the case with the vertical shear forces, as the torsional moment will be carried by both a vertical and a horizontal shear flow (Vayas & Iliopoulos, 2013).

For the shear force, only small changes are expected in the shear connectors. When the bracing systems are applied to the bridge model, the shear force is (marginally) reduced due to a better distribution of the load. However, the torsional stiffness increases the shear flow through the studs. Therefore, only a small variation in the shear flow through the shear connectors is expected in the normal case. On the other hand, for the unloaded girder the relative change of the shear force will increase. Normally, since the shear forces in the shear connector of the unloaded girder are much smaller than the shear force in the shear connector of the loaded girder, this will not be critical for the new static system.

Due to elastic deformation in the diagonals, it is hard to share the load exactly equal between two I-girders strengthened with a lateral bracing. However, a reduction of the stresses in the loaded girder around 20-30 % will however substantially prolong the remaining lifetime of the bridge.

6.3. Suggestions and details for adding lateral bracing in an existing bridge

In the following sections, three different types of connection between the flange and bracing members are suggested. Which type of connection is preferable depends on the existing design and geometry of the bridge. However, the chosen details also depend much on the praxis of the country in which the bridge is located, the opinion/preferences of the designer/steel contractor and so on. Keeping that in mind, the details presented here should be regarded as possible solutions, not as firm rules. Two different connections are necessary for installing the lateral bracing system. A connection between the bottom flange of the existing girder and the bracing member and a connection between the bracing members themselves at the "center of the K-joint, see Figure 38.

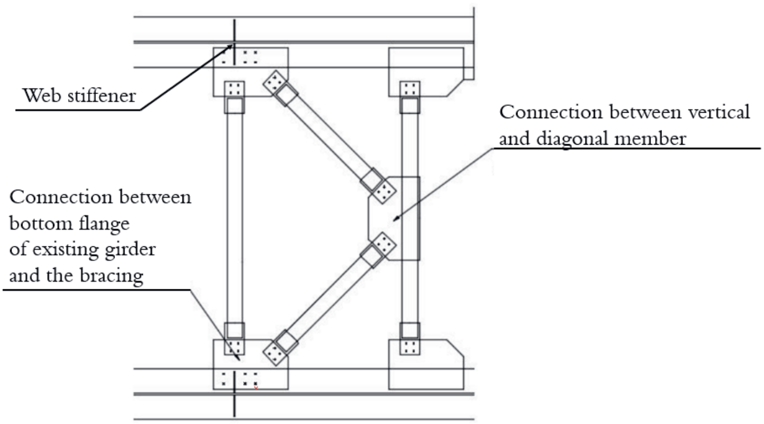

Figure 38 *Suggestion for connecting a K-shape bracing between the bottom flanges.*

Similar types of connections are necessary for the X-shaped bracing system, where the connection between the bracing members themselves is between the two diagonal members. The solutions shown in this section are only shown for bolted connections at the site since welded connections are assumed difficult to produce properly on site and additionally might have a negative effect on the detail class of the bridge and thereby reduce the intended increase of the lifetime of the bridge. By using a bolted connection, the detail class is not reduced, and the trusses thereby have no negative effect on the remaining lifetime of the bridge. Because the new bolted detail will start on a virgin material, in terms of fatigue.

To obtain a robust and solid connection and to avoid/minimize gaps between the gusset plates, all bolted joints are shown with a minimum of four bolts per joint in the following examples. Due to the relatively small forces, it can be argued that the number of bolts can be reduced, e.g., from four to two.

6.3.1. Details for K-shaped bracing

The first suggested solution for the connections in the K-shaped bracing system are presented. In Figure 38 a solution for the connection between the bottom flange of the existing girder and the new trusses is shown. In the design shown it is seen that the trusses are arranged in such manner that the gravity center of the two connecting bracing members coincide with the center of the existing main girder.

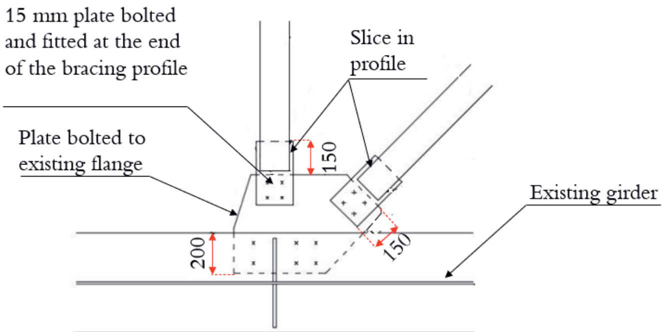

Figure 39 *One solution to attach the bracing members of a K-shaped bracing.*

The connection is constructed as a bolted connection where a common gusset plate is bolted to the lower side of the bottom flange of the existing girder. A plate is fitted through the end of the bracing member and connected with fillet welds on both side of the web of the member. The two plates are connected by bolted connections. All bolts used in the connection are M20 bolts made either as fit bolts or slip resistant (or friction grip) bolts. Tolerances are taken in the bolted connection to fit in the new bracings to the existing conditions.

In the design shown in Figure 38 it is seen that the trusses are arranged in such manner that the gravity center of the two connecting bracing members coincide with the center of the existing main girder.

In Figure 40 the suggested solution for the connection between the transverse and diagonal bracing member in the K-shaped bracing system is shown.

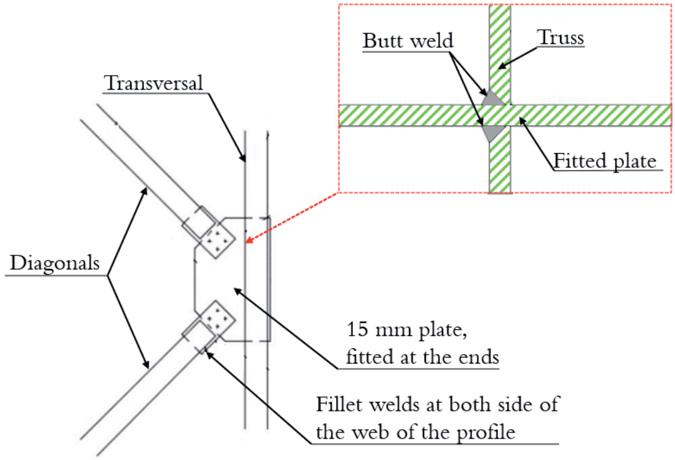

Figure 40 *Connection between the diagonal and the transversal member.*

As for the connection between flange and bracing members the suggested solution for the connection between the transverse and diagonal bracing is constructed by sliding a plate through the transverse truss. The connection between the plate and the transverse truss are prepared with butt welds, where the web of the bracing is beveled so the butt weld can burn through. As for the connection showed in Figure 40, a plate is split through the end of the truss and connected to the truss with simple fillet welds. All bolts shown are M20 bolts designed either as fit bolts or slip resistant bolts.

In the solution for the connection showed in Figure 39, the total number of bolts used is relatively large. Since the cost is strongly dependent on the time used for installation, an alternative solution is proposed, where the transverse bracing is welded directly to the plate that is bolted to the flange. Thereby the number of bolts needed is reduced from 14 to 10, see Figure 38 and Figure 41. Also, in the design shown in Figure 41it is seen that the bracing members are arranged in such manner that the center of gravity of the two connecting members coincide with the center of the existing main girder.

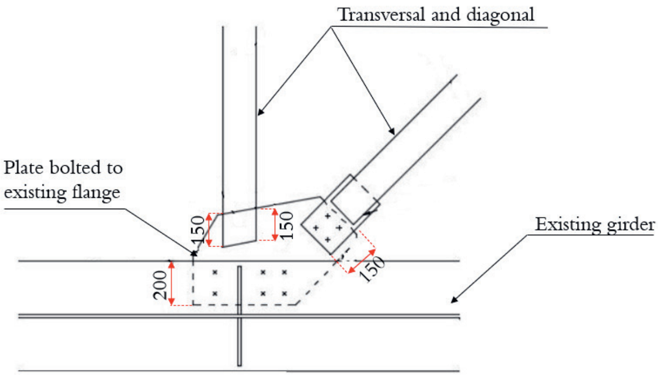

__Figure 41__ Additional solution to attach the bracing members of a K-shaped bracing.

The downside to this solution is that there is less room for tolerances since all tolerances are now taken in the diagonal. The connection between the plate and the transversal member is necessary to be welded before installing the bracing. Therefore, the weight of the element consisting of plate and transversal, that needs to be bolted to the existing flange, is higher than for the solution in Figure 39.

To reduce the number of bolts needed for the connection, but still obtain the same possibility for fitting and approximately the same weight of the different parts that need to be installed a third solution is suggested, see Figure 42. In this solution for the connection between flange and bracing, the small plate fitted at the end of the bracing members is bolted directly to the bottom flange. The large gusset plate is thereby avoided and the total number of bolts necessary to make the connection is reduced from 14 to 8. At the same time the total weight of the added steel is reduced. Tolerances can be handled in the bolted connections. The angle (α or β) of the diagonal can easily be changed to fit the existing conditions off the bridge.

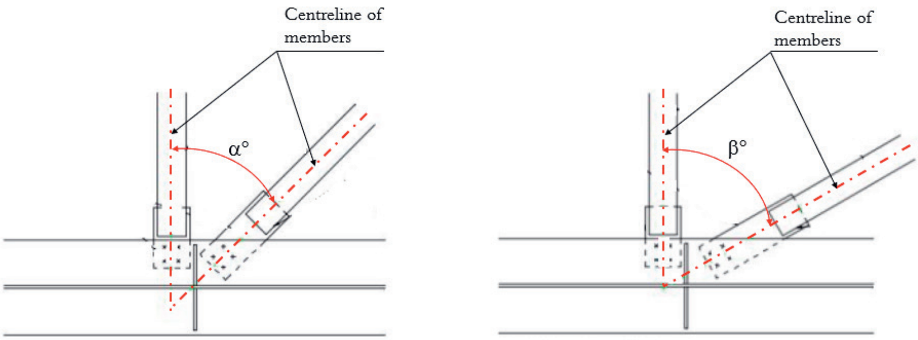

__Figure 42__ Additional solution of a K-shaped bracing with different angles.

The downside of using this solution is that the centerline of the transversal and diagonal (shown by the red dotted line in Figure 42) does not intersect at the position of the web stiffener. This means that the center of the connection is not placed at the position of the web stiffener and therefore this connection has a larger risk of introducing additional local stress increase around the connection than the solutions shown in Figure 39 and Figure 41. Furthermore, in the case where the angle of the diagonals is larger than shown in Figure 42, it might be impossible to design the point of intersection of the two truss members at the centerline of the main girder.

6.3.2. Details for X-shaped bracing

The suggested solution for the connection between the bottom flange of the existing girder and the bracing members for the X-shaped bracing system is designed using the same principles as for the K-shaped system. The only modification is that an additional diagonal is added whereby the connection is symmetric around the centerline of the transverse member. In Figure 43 one suggested solution for the connection between the two diagonals for the X-shaped bracing system is shown.

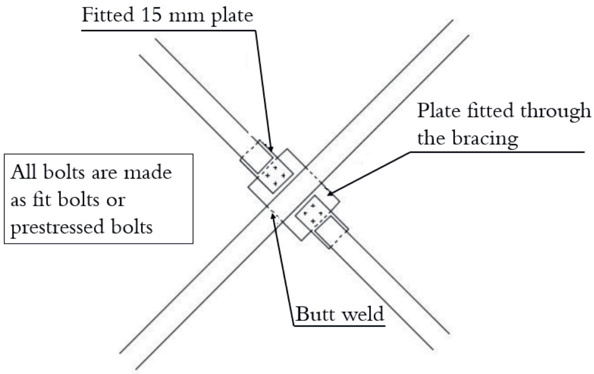

Figure 43 *Proposal for the connection between the diagonals in the X-shaped bracing system.*

The suggested solution for the connection in the X-shaped bracing system is made as for the K-shaped bracing system. One of the diagonals is continuous while the other is cut in half. The connection is made by sliding a plate through the continuous diagonal and connection the other diagonal by bolted connections.

7. Discussion, conclusions and suggestions for future research

7.1. Discussion and answers to the research questions

In the beginning of this thesis, Ch 1.1, five research questions were stated with respect to the overall objectives and aim of this project. Discussion and answers (RA) to the research questions (RQ) can be found below:

RQ1. *To what extent has the concept with lateral bracings between the bottom flanges been used in new, and existing bridges, around the world?*

RA1. The use of lateral bracing in bridges has been studied in Paper I (Vestman et.al. 2023) appended to this thesis. It was found that in the countries included in the study, only one has regulations regarding the use of lateral bracing in I-girder bridges. Finland has regulations stating that lateral bracing should be used if the bridge has a longer span than 50-70 m (Liikenneviraston, 2016). Additional to these regulations, the general recommendation in practice is that this rule should be implemented for any bridge with spans longer than 70 m. Furthermore, the other countries included in the study have composite I-girder bridges with lateral bracing between the bottom flanges.

In France, the Porte Sud Bridge is a good example of where this concept could be competitive in comparison to the more commonly used box-girder system. For this specific case the final solution of I-girders with lateral bracing between the bottom flanges saved around 15 % of the total steel weight compared to a box-girder solution (Berthellemy, 2002). In the US, bridges with two main girders (twin girder solution) are rare. Several bridges with multiple girders have lateral bracing between the bottom flanges of the main longitudinal girders. For these cases it is more of a question for which areas (bays) the lateral bracing should be implemented, rather than if it should be implemented at all.

In Paper III (Vestman et.al. 2022) an existing bridge in Norway was studied, where the lateral bracing most likely was intended as a stabilization system for horizontal loads, as wind. It was found that the lateral bracing, which had relatively slender members, need to be considered for the additional member forces from the torsional moment of the quasi-closed cross section. This will also certain be the case for other existing bridges where the main purpose of the lateral bracing has not been to distribute eccentric vertical loads, which means that additional forces from the torsion may have been overlooked in the design.

It can be concluded that the concept of lateral bracing between bottom flanges has been used to distribute both horizontal and vertical loads in composite bridges in some countries. The author has however not found the use of any example, in the literature, where the concept has been used for strengthening composite I-girder bridges.

RQ2. *What potential benefit can the concept provide for a new composite I-girder bridge, compared to an I-girder bridge without lateral bracings or a bridge with a box cross-section?*

RA2. One of the biggest benefits for an I-girder bridge by implementing a lateral bracing is the more torsional stiff cross section, which enables a better load distribution for eccentric loads. This means that the longitudinal stresses from eccentric loads, for example the fatigue load model, are decreased. This could mean a lot for the design where the fatigue limit state, FLS, is the governing case for some sections of the steel girders. A reduced fatigue stress would mean that less material should be needed to fulfill the requirements.

Even if it would be proven that a composite I-girder bridge would be comparable to a composite box-girder bridge in terms of load distribution of eccentric loads, it does not necessary mean that they are comparable as concepts. Further factors as esthetics, tradition in design and regulations need to be considered. However, the concepts are similar in term of the technical function of resisting and distributing the design loads.

In Paper I (Vestman et.al. 2023) the benefits of using I-girders with lateral bracing compared to box-girders regarding the manufacturing was discussed. The main purpose of using I-girders instead of a box-girder is that it simplifies the welding- and operation procedure at the steel workshop. Furthermore, the I-girder system will enable the parts to be handled and transported in more manageable sections, compared to a uniform compact box-girder section.

RQ3. *How much of the eccentric traffic design load could approximately be redistributed to the least loaded girder, by implementing lateral bracing on a typical Swedish composite bridge?*

RA3. The distribution of eccentric loads will be improved by implementing lateral bracing between the bottom flanges of two I-girders in a composite bridge. Generally, the impact on the load distribution from the more torsional stiff cross section (quasi-closed cross section) has been a reduction of 20-30 % of the load for the most loaded girder, in the studies that have been conducted. In both Paper II (Vestman et.al 2018) and Paper IV (Vestman et.al. 2022) it was found that two existing Swedish bridges could benefit from lateral bracing. In these studies, both a continuous span- and a simply supported bridge were analyzed. The results proved that the load distribution was improved from a ratio of 0,95/0,05 to 0,70/0,30, where the impact ratio was dependent on the stiffness of the bracing members rather than the shape of bracing.

Theoretically, it would be possible to reach a load distribution ratio closer to 0,5/0,5, which is a limit value that cannot be reach for an open section. Since the load distribution is depending on more than just the implementation of the lateral bracing, for example elongations of the transversal members of the bracing, a more realistic limit of the load distribution could be around 0,7/0,3. This means that moderate structural members (steel profiles) are used in the bracing and that the concrete deck, which stiffness certainly have an impact, also has realistic properties.

RQ4. *How will the existing structural elements, such as cross frames and the concrete deck be affected by adding a lateral bracing?*

RA4. The impact from adding a lateral bracing on existing structural elements has been evaluated in Paper (II–V), where additional effects on other structural elements than the main girders were specially investigated in Paper IV (Vestman et.al., 2022). It was found that the internal cross frames were affected by increased normal forces in the members when a lateral bracing was added. In this specific case the normal forces were increased around 50 kN, corresponding to a normal stress of 25 MPa. This may not seem a lot, but in terms of fatigue stresses this could be severe if the detailing of the connection joints of the cross frame are poorly designed. As for the case where the lateral bracing forces may have been overlooked in the design (Paper III), particular concern of the impact on the existing cross frame is needed when a lateral bracing is added.

In Ivanov, Collin & Vestman (2020) the impact on the concrete deck from transforming the open I-girder cross section into a quasi-closed cross section was studied. From the results it was shown that the impact mostly was for the better for the concrete deck, but some additional effects need to be considered further. These effects are for the change in shear flow at the steel-concrete interface. In the case of the quasi-closed cross section, the vertical bracings resist the distortion of the cross section by maintaining its shape. This leads to additional loading of the shear connection in horizontal direction at the location of the internal cross frames. In Paper IV (Vestman et.al. 2022) it was concluded that the decrease of the shear flow at the steel-concrete interface was around 10 %, which could be compared to the reduction of the global bending stresses and displacements of 22 % for the same case. This is however expected, since even for a closed box girder with eccentric loading, the webs will have substantially different shear forces (even if the displacement of the two webs in the cross section is almost the same). This is due to the fact that the webs will get an additional contribution of the total torsional moment from eccentric loading acting on the cross section.

RQ5. *How could the lateral bracing be designed regarding its connection details, from design and production aspects and how could it be connected to the existing structure?*

RA5. In this thesis some proposed ways of implementing a lateral bracing is presented in Ch. 6 for K- and X-shaped bracings. Within this chapter, some possible details for the connection joints are described and illustrated. These should not be seen as optimal solutions, but rather possible solutions where factors of importance for assembling and structural aspects are included. The preferred way of installing the bracing on an existing bridge would be with connections bolted to the bottom flanges. This facilitates the assembling procedure and gives a better resistance to fatigue than what an on-site welded connection would give. The bolted connections could preferably be designed with prestressed bolts and as double lap joints. The shape and configuration of the bracing need to be determined from case to case, where the bracing and the connections can be optimized.

7.2. Future research

Regarding future research within this subject, some suggestions of possible topics are listed below:

- Investigate the impact on the torsional behavior, for a composite I-girder bridge with lateral bracing between the bottom flanges, from the modelled stiffness of the shear connectors at the steel-concrete interface. As mentioned above, the shear flow from torsional moments will increase in the horizontal direction at the location of internal cross frames. If the shear connectors are modelled as rigid, this increase will have a high peak value at these locations, which probably is not the case in reality. A less rigid stiffness of the connectors would capture a more realistic behavior and lower the stress peaks of the shear flow.

- In this thesis a comparison of the structural behavior and some other aspects for I-girder bridges with lateral bracing and box-girders has been done. Further, these systems could be compared regarding the economic benefits. A concept bridge could be used, both, to compare a bridge concept with two I-girders, I-girders with lateral bracing between the bottom flanges or a box-girder.

- Since "the devil is in the details", it would be valuable to strengthen an existing bridge by using the concept presented in this thesis and monitor it both before and after the strengthening.

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Paper I

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Horizontal bracing between bottom flanges in composite I-girder bridges – A State of the Art Review

Horizontal bracing between bottom flanges in composite I-girder bridges – A State of the Art Review

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Abstract

The tradition in how steel bridges are designed and built varies around the globe. These traditions can be based upon the accessibility to materials and products, but also the topographical conditions for the bridge locations. In Nordic countries, welded steel plate girders are the most common type of steel bridge superstructure. In contrast, many other European countries construct their steel bridge superstructures with rolled steel sections. Variations can also be seen in how certain details are designed and detailed, such as preferences for welded or bolted field constructed joints. The differences in the use of a horizontal bracing between the main girders in different countries are not well known. The bracing is often used to distribute horizontal loads, such as wind loading, but some countries also count on this horizontal bracing to increase of the torsional stiffness of the structure and better distribute eccentric vertical loads, as well.

This paper investigates the premise that the use of horizontal bracing between the bottom flanges of new I-girder bridges results in a cross-section that behaves similarly to a steel box-beam where the vertical webs share more equally in the applied loads. This concept lowers the beam stresses and is particularly beneficial for fatigue evaluation caused by eccentric loads. The concept is exemplified by in-service

bridges in Finland, Guatemala, France, and the USA. The potential to use horizontal bracing as a strengthening technique in existing steel two-girder composite bridges is also discussed.

Keywords: **bridge; composite bridge; horizontal trusses; horizontal bracing; lateral bracing; strengthening; torsional stiffness**

Introduction

Background

The use of horizontal bracing, also known as lateral bracing, is not consistent throughout the world and the regulations and requirements around its use are quite varied. The authors of this paper have neither found any compilation nor many papers on the use and experience of trusses transforming composite I-girder bridges into closed form sections. One reason may be that practicing engineers, in many cases, do not have the time or incentives to turn their bridge projects into conference/journal papers but this paper, mainly based on the experiences of the authors, might be useful to spread knowledge about this concept. Some countries specifically require the use of horizontal bracing while others prohibit their use. The reasons for this disparity can be traced back to the type of structures that the country typically constructs. For example, in Nordic countries, bridges are comprised of only two welded steel girders. These are very efficient for spanning over rivers with steep embankments where there is no limitation for the free height under the bridge. These girders might benefit from horizontal bracing during construction and while in service to alleviate stability concerns. In contrast, bridges in other countries are often constructed with a bridge cross-section consisting of multiple rolled steel beams. These types of bridges are often more resistant to stability issues such as lateral torsional buckling (LTB) and do not benefit from the use of horizontal bracing until the span lengths become quite long or have a small horizontal curvature.

This paper explores how horizontal bracing is used around the world to distribute loads between primary longitudinal members, provide resistance to lateral loads, and could permit an existing two-girder structural system to be retrofitted to behave similarly to an often more expensive closed box system.

Horizontal Bracing in Bridges

In addition to the variation of use, the truss shape for the horizontal bracings also varies. Some of the most common types of shapes are illustrated in Figure 1.

- K-truss
- D-truss (also called Warren type)
- X-truss
- Z-truss (also called Pratt type)

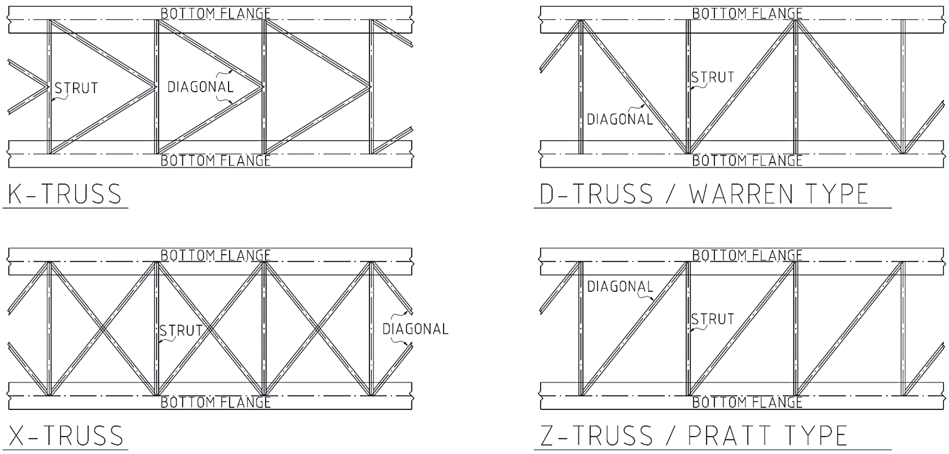

Figure 1. Different types of shapes for the truss.

Impact of Horizontal Bracing on I-Girder Bridges

For a symmetric bridge with two steel I-girders supporting a composite concrete deck, the dead load is normally assumed to be evenly distributed between the girders. An eccentric load, be it superimposed dead load or live load, will be unevenly distributed with the girder closest to the load carrying more than the other girder. In contrast, box-girder sections more evenly distribute eccentrically positioned loads due to the higher torsional stiffness of the box-girder section. This is true provided that the box-girder section has sufficient internal intermediate cross frames or diaphragms to prevent distortion of the cross-section which would lead to out of plane bending stresses in the webs and full-width bottom flange. In spite of their structural advantages, I-girders are often preferred because box girders are typically more expensive to fabricate, ship, and erect than individual I-girders.

To achieve higher torsional stiffness for an open cross-section like steel I-girders with a composite concrete deck, some countries permit the use of horizontal bracing between the main steel girders. The purpose for increasing the torsional stiffness varies from case to case and country to country. In some cases, it is due to the safety and reliability of the structure while in others it is due to regulations of the behavior of the bridge in the service limit state (Liikenneviraston 2016). The horizontal bracing is placed as a truss between the lower flanges, such as a K-shaped pattern along the girders. The horizontal bracing changes the behavior of the bridge, in terms of torsional stiffness, to replicate that of a box section.

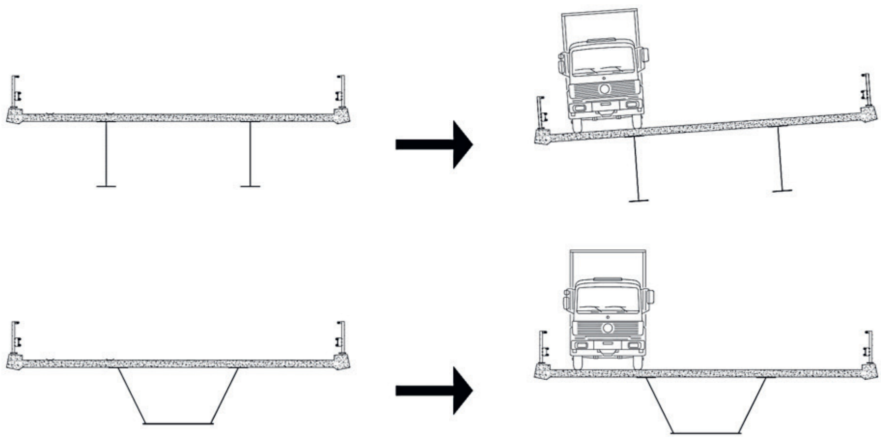

Figure 3. A schematic sketch of the deflection due to eccentric load on both an I-girder and box girder cross-section (excluding any warping effects).

The uneven distribution of live load between the I-girders affects the Ultimate Limit States (ULS) but affects the Fatigue Limit State (FLS) even more. The FLS is typically defined in terms of accumulated significant stress-range cycles within specific structural details. Typically, only heavy truck loads, especially eccentric truck loads, generate the significant stress-range cycles. Certain welded connection details are more fatigue sensitive than others. Where these details cannot be replaced with more fatigue tolerant details, larger steel member sizes are required to lower the stress range enough that the FLS doesn't control the design.

After the introduction of the Eurocodes throughout Europe, many countries instituted stricter requirements for fatigue of steel members. For example, a Swedish bridge with steel I-girders and a composite concrete deck designed under the new fatigue requirements requires bottom flanges at midspan with almost twice the steel area than what was required by the previous Swedish bridge code.

Case studies have shown that a way to reduce the stress-range cycles of a twin structural steel I-girder bridge with a composite concrete deck is to increase the torsional stiffness of the system by the addition of horizontal bracing between the bottom flanges (Vestman et al. 2018), (Vestman et al. 2016) and (Collin et al. 2018). These case studies verify that the increase of torsional stiffness increases the distribution from eccentric loads and lower the stresses in the most loaded girder, which increases the remaining fatigue life of the bridge.

With respect to torsion, the bottom flange horizontal bracing makes the flanges and bracing act as a homogeneous flange, as in a steel box-girder (Vavas and Iliopoulos 2013). The increase of the torsional stiffness can be illustrated by the difference between the torsional stiffness of an open and a closed cross-section, as illustrated in Figure 4. Closed cross-sections carry the torsion induced by eccentric loads through shear flow around the entire cross-section. The shear stresses (τ) shown in Figure 4 are caused by the St. Venant component of the applied torsional moment (M_T). The other component from the applied torsion is the warping torsion. This component is dependent on the stiffness and placement of the internal cross-bracing system, with vertical trusses/beams connecting the I-girders. A too widely spaced internal cross-bracing system with cross-girders or diaphragms, could lead to a severe distortion of the cross section.

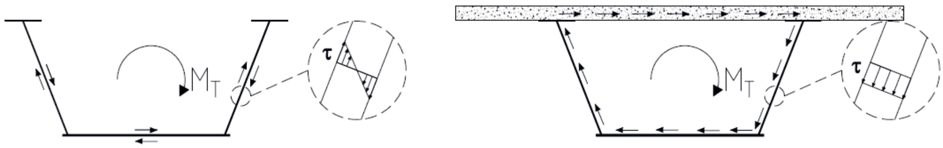

Figure 4. Shear stress from St. Venant torsion in an open- and closed cross section.

The shear flow will be carried as axial forces in the horizontal bracing system between the bottom flanges. To determine the effective cross-sectional properties of the combined twin I-girders, the horizontal bracing can be represented as an equivalent fictitious plate thickness. By using the virtual work principal, the shear stiffness of the bracing and the shear stiffness of a plate can be compared to determine the fictive thickness of the bracing system expressed as a plate between the bottom flanges (Roik 1983) and (Kolbrunner and Basler 1969). The shear flow must then be transformed to forces in the bracing members using statics, as described later in this paper.

For existing bridges, adding horizontal bracing could reduce the stress-range cycles enough to substantially increase the remaining fatigue life of the structure. For a twin steel I-girder composite bridge the load effect from eccentric live loads have often been calculated with the assumption that the concrete deck is simply supported between the two girders. Using a simple statics analysis, this results in a load effect factor greater than 1.0 when the load is placed on the cantilevers of the concrete deck. Even using a more precise method, such as a 3D finite element (FE) analysis, the load effect factor will be shown to be around 1.0. For the reviewed case studies, the load effect factor for the most eccentric load case is approximately 1.2 when a simply supported deck is assumed and 0.95 when a 3D FE analysis is used (Vestman et al. 2018), (Vestman et al. 2016) and (Collin et al. 2018).

When a horizontal bracing system was added to a twin steel I-girder bridge with a composite concrete deck, the increased torsional stiffness of the cross-section for the bridge in (Collin et al. 2018) improved the load effect factor from 1–0.95 to approximately 0.70, as illustrated in Figure 4. The corresponding 30% reduction in live load stresses increases the remaining fatigue life by approximately a factor 6, with a slope of m=5 in the Wöhler curve (EN 1993-1-9 2010). By increasing the remaining fatigue life, bridge owners may be able to delay further fatigue mitigation investments of the structure well into the future.

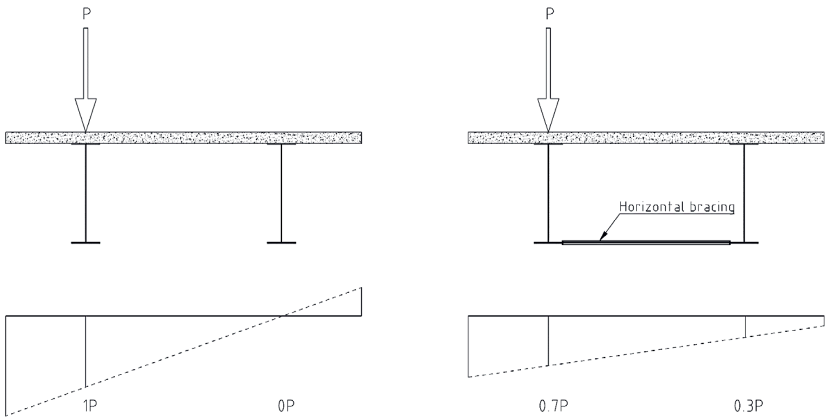

Figure 5. Distribution of an eccentric load (P) for a twin I-girder composite bridge without and with a horizontal bracing.

Design Methods

The design of the cross-section has traditionally been carried out by transforming the horizontal bracing to an equivalent bottom plate. The fictive bottom plate thickness, t^*, for a K-truss as in Figure 1 and Figure 2, could be calculated with the following equation (Roik 1983).

K-shaped truss:

$$t^* = \frac{E}{G} * \frac{ab}{\frac{2d^3}{A_D} + \frac{b^3}{4A_S} + \frac{a^3}{12}\left(\frac{1}{A_F} + \frac{1}{A_F}\right)}$$

(Eq. 1)

Where:

t^* = Equivalent bottom plate thickness

a = Distance between the vertical beams in the framework

b = Distance between the girders

d = Length of the diagonals in the framework

A_S = Cross-sectional area of the strut

A_D = Cross-sectional area of the diagonal

A_F = Cross-sectional area of the bottom flanges of the girders + a contributing part of the web area

E = Young's modulus of steel

G = Shear modulus of steel

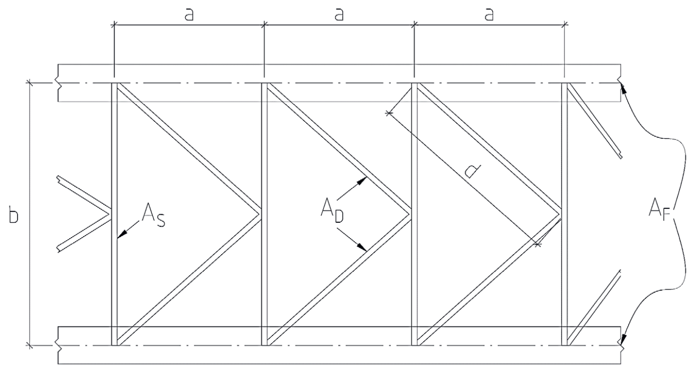

Figure 2. Geometry of the K-truss used in the eq. (1)

A grillage model can be used to analyze the global behavior of a steel girder bridge by substituting the thickness obtained from Eq. 1 as the thickness of an equivalent bottom flange between the girders. The cross section can then be analyzed as a closed section to obtain the lateral forces. These lateral forces can then be resolved into the individual member forces using statics.

Recent developments in FEM software allow the modeling of the structure with shell elements, so the horizontal and transversal bracings can also be included in the model. This allows direct results, but also puts more responsibility on the designer to verify the results.

Due to the challenges in the modeling and analysis, some of the finer aspects of the design of the bracing members and their connections may have been overlooked for bridges designed before more advanced calculation methods were available. Given that there are no widespread reports of fatigue problems, it seems that the structural details are adequate for their intended purpose.

International Experiences Using Horizontal Bracing

Finland

The concept of sturdy horizontal bracing has been used in Finland for large steel beam bridges since the late 1970s. Lighter steel beam profiles with an open cross section, whose main purpose is stiffening against horizontal loads during erection and service, were already in use well before the 1970s.

The main functions of sturdy horizontal bracings, usually constructed from hollow rectangular steel sections (RHS), are to stiffen the structure against horizontal loadings and to add torsional stiffness to the structure. In the first bridges constructed with horizontal bracings, they were mainly used to enhance torsional stiffness. Two such bridges were the Tervola bridge, built in 1975 with a main span of 236 ft./72 m) and the Kaitainen, bridge built in 1982 with a main span of 90 m (295 ft.). Currently, Finland has about 35 steel girder bridges with horizontal bracing. National guidelines in Finland advise that the use of horizontal bracing is advantageous for span lengths exceeding 50 m (165 ft.) – 70 m (230 ft.) (Liikenneviraston 2016). In practice, this advice is often taken as a requirement and any bridge span exceeding 70 m (230 ft.) automatically uses horizontal bracing.

Figure 6. Kaitainen bridge.

Another Finnish bridge example is the Jännevirta bridge built in 2018 and is one of Finland top ten longest bridges. The bridge is almost 600 m (2000 ft.) long and has a deck width of 15 m (49.2 ft.). The longest span is 120 m (393 ft.) and the superstructure is built with steel I-girders that haunch down at the intermediate supports. Both the Kaitainen bridge (Figure 6) and the Jännevirta bridge (Figure 7) utilized a K-shaped truss for the horizontal bracing system.

Figure 7. Jännevirta bridge

The horizontal bracing increases torsional stiffness and redistributes the live load effects between the main beams and decreases transversal inclination of the deck caused by eccentric loading. This reduction in live load stress has a clear influence on the load carrying capacity and an especially large influence on the fatigue capacity of the girder (Collin et al. 2018).

In Finland, the most frequently used type of horizontal bracings between I-girders is the K-type bracing. It has been found that K-type horizontal bracing is advantageous because, in the global behavior, it does not act as a bottom flange in the main girders (Vestman et al 2018). Also, since the system nodes coincide, this type of bracing does not induce secondary stresses in the main girders. Similarly, the vertical transversal bracing is of a single K-type truss, opening upwards. The top chord of the transverse truss is usually temporary since the composite concrete deck can take the required loading after curing. If the top chord is not removed, it must be designed to resist the concrete slab shrinkage forces. If the distance between the main girders is large, a single K-type truss is not possible, and one or a pair of K-type diagonals are added (Figure 8).

Figure 8. Cross bracings in the Mälkiä channel bridge A

In such a case, the top chord cannot be removed. Normally, a horizontal gusset plate (penetrating the bottom chord) is used to connect horizontal diagonals (Figure 9).

Figure 9. Connections between the horizontal and vertical bracing at Vekaransalmi Bridge.

USA and Latin America

Design Practice and Considerations in AASHTO LRFD

In the US and most Latin American countries, the main bridge design code is the AASHTO LRFD Bridge Design Specifications (LRFD). This code is used in some specific types of bridges, and it provides simplified rules valid only for a limited range of geometries and structural configurations.

LRFD deals with two types of composite bridges: I-girder and box girder steel and concrete composite bridges. I-girder bridges are usually composed of more than two girders. The reason is that in a bridge with only two main girders, the longitudinal girders are considered fracture critical members. Fracture critical members require more fabrication quality control and design checks than non-fracture critical girders. Bridges with three or more girders do not require the additional fabrication and design checks. Figure 10 shows an example of a fully integral abutment bridge using multiple curved weathering steel girders and bottom horizontal bracing under construction in the NY City area. The composite concrete deck is placed on foam filled stay-in-place forms and the bottom lateral bracing is used only in the exterior bays.

Figure 10. Horizontal bracing located only in the exterior bays and stay-in-place forms visible between the girders.

Figure 11 shows an eight-span multiple curved weathering steel girder bridge that will carry Route 17 WB over the Chenango River near Binghamton, NY. The girders are continuous for four spans with multi-cell modular joints at each abutment and near the middle of the structure at Pier 4.

Figure 11. Bridge during the construction stage. The multi-cell modular joint is just out of view to the right.

A survey was circulated to all 50 Department's of Transportation in the USA asking about the requirements that each state had concerning horizontal bracing. Of the 30 agencies that responded, only 3 agencies had provisions that required the use of horizontal bracing. For straight girders, 2 agencies encouraged the use of horizontal bracing for spans greater than 43 m (149 ft.) but would permit their exclusion if calculations showed that they were not necessary. For curved girders, 1 agency required use of lateral bracing regardless of the degree of curvature. It should then be no surprise that the standard details from the Federal Highway Administration do not show the presumptive use of horizontal bracing, but provides useful rules and recommendations (Federal Highway Administration 2020). In fact, the most widely used software programs for the design and rating of steel bridges (ex. MDX and LEAP), do not consider the structural contribution of lower horizontal bracing in their analysis.

Oxec Bridge, Guatemala

In spite of the fact that bridges designed using LRFD steel bridges do not often use horizontal bracing, the bridge design company Pedelta has recently designed some bridges with lower horizontal bracing in Central America that meet AASHTO LRFD requirements.

The Oxec Bridge, recently built in Guatemala, is an example of a bridge with lower horizontal bracing (Figure 12). The bridge is on a constant grade of 1% and is composed of three tangent spans with lengths of 49.5 m (162 ft.), 77 m (252 ft.), and 49.5 m (162 ft.) for an overall length of 176 m (577 ft.). The structure was designed and checked to be incrementally launched from one end. The superstructure consists of a composite concrete deck supported by three continuous 3.2 m (10.5 ft.) deep steel I-girders spaced at 3.4 m (11 ft.). The total deck width of 9.6 m (31.5 ft.) accommodates two traffic lanes, lateral shoulders, and a steel traffic barrier in the middle.

Figure 12. *Oxec Bridge in the workshop.*

Since the cross-section only had three girders, the designer proposed to use horizontal bracing for both the top and bottom flanges to increase the torsional stiffness and to have further redundancy and robustness during the launching stage. This was the first launched bridge in the country, and the designer wanted to minimize the risk of damage or collapse during the launching process. The designer designed redundant resisting mechanisms so that if some of the members failed due to unanticipated conditions, the system would remain stable. For example, the designer considered that in some operations a gap could exist between the lower flange of the beam and the lateral guide which could result in the failure of some lower bracings or cross-frame diagonals and ensured that there was enough remaining capacity for the system to remain stable.

A global three-dimensional finite element model (shell and beam elements) that included the lower horizontal bracing was used to ensure that the forces and stresses in the steel members once the bridge was opened to traffic remained within allowable values. The analysis of the bridge in its final condition and open to traffic was performed using a simplified analysis considering the transverse frames as equivalent beams by means of a commercial software package that included all the Code verifications and requirements.

Bridge 15 over Pinulla River, Guatemala

Bridge 15 spans the Pinula River in the municipality of Villa Canales and is part of the new South access road to Guatemala City. The bridge was designed to be incrementally launched. One of the first launching sequences of the bridge is shown in Figure 13 and the almost finished launching of the bridge is shown in Figure 14.

Figure 13. Initial launching of bridge 15 over Pinulla River

Figure 14. Launching nearly completed.

Bridge 15 has an overall length of 528 ft./161 m, divided into three remarkably uneven spans of 51 m (167 ft.) + 50 m (164 ft.) + 60 m (199 ft.). The bridge is entirely curved in plan, with a curvature radius of 485.5 ft./148 m, has a constant grade of 3.2%, and has a superelevation of 7.6%. The superstructure consists of six 3.0 m (10 ft.) deep girders, spaced at 3.2 m (10.5 ft.), supporting a composite reinforced concrete slab of 275 mm (10.75 in.) thickness. The deck has concrete barrier on each fascia and accommodates four lanes of traffic (two in each direction) with a concrete median barrier for a total deck width of 17.96 m (59 ft.) (Pedelta 2021).

The tight in-plan curvature of the bridge, along with the necessary pre-camber of the steel structure, cause certain areas of the deck to come off the temporary bearings during several launching stages and makes the distribution of reactions on these temporary bearings and the deck itself difficult to estimate. To accurately obtain these reactions, a 3D nonlinear model of the type "composite" was used. In the model, bar elements were used for the top and bottom flanges, shell elements were used for the webs, and the members of the cross-frames and horizontal bracing used beam elements.

Two different configurations for the bracing were analyzed. Firstly, the upper and lower bracings in the top and bottom flange of the girders were analyzed. Very high internal forces in the bottom horizontal bracing were found, which required large member sizes and would make this design more expensive and complex compared to a design without lower bracing. In this bridge, the high curvature of the bridge leads to undesirable effects during launching. The lower bracing dramatically increased the torsion stiffness of the bridge and made it almost impossible to accommodate the bridge deflections over the temporary bearings during the launching operations including pre-cambering. When the lower bracings were removed from the analysis, the longitudinal girders were able to deflect more independently from each other, and the uplift forces during launching over the temporary bearings at the launching yard were much lower. In other words, the reactions at the temporary bearings were more evenly distributed at each support line. For these reasons a design without lower bracing in most of the length of the bridge was chosen. Lower horizontal bracing was still used close to the diaphragms at piers to provide resistance to the seismic horizontal loads.

France

French Experiences for Rail and Road Bridges

In France twin girders with bracings are commonly used for bridges carrying two tracks for high-speed train bridges. The increased torsional stiffness associated with the use of the bottom lateral bracing is useful to limit the girder deflections due to eccentricity when a train uses the bridge. The reduced deformation translates into reduced horizontal acceleration and the ability to easily satisfy the comfort rules can be satisfied. Increasing the bending stiffness of each beam to achieve the same reduction in deflection would be more expensive.

For road bridges, the use of bottom horizontal bracing was implemented only once in Lille at the Porte Sud, linking the A1 and A25 motorways and the Boulevard Périphérique Est of Lille. The Porte Sud interchange is one of the busiest in France. This strategic bridge is part of the eastern bypass of Lille and provides a link between the A1 motorway (Paris) and the A25 motorway (Dunkirk) which also leads to the Channel Tunnel.

The length between roadway joints is 289.50 m (950 ft.). The structure has 6 spans defined as follows (see also Figure 7): 36 m (118ft.) – 48 m (157 ft.) – 60 m (200 ft.) – 48 m (157 ft.) – 54 m (155 ft. –42 m (138 ft.). The longitudinal profile is a parabola with a radius of 3000 m (9,842 ft.). The cross-section is made up of two 3.50 m (11.5 ft.) lanes. The constant superelevation is of 2.5%. In plan, the structure is curved with a radius of ´304.50 m (1,000 ft.) at the axis.

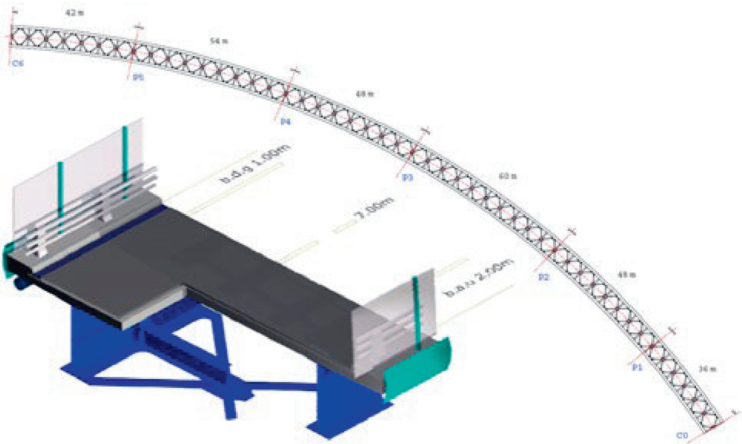

Figure 15. Top view of the bridge

At the time, French designers often considered box girders as the solution for curved bridges. This is true for prestressed concrete bridges because of the useful uniform torsional stiffness, but curved steel box girders are much more expensive than straight steel box girders.

Unsurprisingly, the first design of the Lille Porte Sud bridge resulted in a very expensive box girder. It was redesigned as a twin girder bridge with horizontal bracing to reduce the effects of the eccentricity of the Eurocode loads on the resistance and fatigue calculations. It was also decided to design the bridge for the Class 1 of traffic of the upcoming edition of the Eurocodes, which were only draft at the time. By switching from a box girder to a braced twin-girder, the total mass of the structure was reduced by 15% and the cost of fabrication was reduced by 40% per pound/kilogram.

Characteristics of the Structure and the Bracing

The main beams are 2.2 m (7.2 ft.) deep and spaced at 7.0 m (23 ft.). The distances between the bracing attachments were standardized at 6 m (20 ft.) over the entire structure. The Porte Sud bridge was completed in 2001 and required 830 metric tons (915 tons) of steel, including the bracings. The beams were fabricated from welded plates and the bracing consists of WT sections. To greatly reduce the risk of buckling of the angles, a K-bracing arrangement was preferred over a X-truss shape (Figure 16).

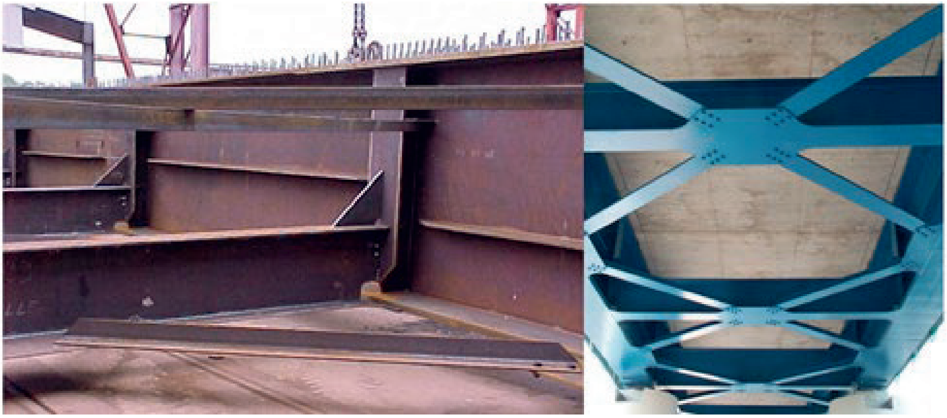

Figure 16. View of the bracings during construction (left) and for the finished bridge (right).

The double girder structure with lower horizontal bracing is similar to an open U-section with the center of torsion below the center of gravity. Under these conditions, the structure is unstable and the use of temporary upper bracing to obtain a closed structure was required.

Design of the Structure

The calculations of the structure were carried out using several calculation models according to the desired checks. Two types of methods were used to determine the deflections. One of the methods was a grillage model that included the horizontal bracing, represented by a fictive bottom flange. The other method was with a refined 3D model including all the structural parts represented by shell and beam elements. Both methods accurately predicted the resulting deflection of the structure.

Conclusions

This paper highlighted some I-girder bridges with bottom horizontal trusses that introduce torsional stiffness. Using trusses to achieve torsional stiffness and simulate a box-like behavior is beneficial in resisting eccentric loading, especially for the Fatigue Limit State. Fabrication, transportation, and erection costs of actual box-girders make the bottom lateral truss a potentially cost-effective alternative.

Some differences in tradition and design of steel-concrete composite bridge around the world are demonstrated. Much of the design principles can certainly be derived in the accessibility of steel products. For countries located near a steel plant making long products (rolled sections), a bridge structure with multiple girders of rolled sections is more natural than using larger welded girders. Also, the shape of the landscape has an impact on the choice of structure, where the topography in the Nordic countries with its steep rivers slopes differs from the flatter landscapes found in areas such as the Netherlands.

If a permanent or temporary horizontal truss between the top flanges is used, it also gives a high torsional stiffness before the concrete deck is cast, which can help during the launching of the bridge. The permanent or temporary horizontal trusses also increase structural stability during concrete deck placement when there is a high wet concrete load but no concrete deck strength.

The concept also has potential to be used for strengthening of existing I-girder bridges with fatigue loading concerns. The horizontal bracing truss is preferably bolted to the bottom flange to make the field-assembly of the bracing system more efficient and flexible, and to implement a new fatigue detail on virgin material in terms of fatigue. This strengthening concept has been investigated in the European RFCS-project ProLife (Collin et al. 2018).

Acknowledgment

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Paper II

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Box-action giving new life-time to old steel bridges

Box-action giving new life-time to old steel bridges

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Abstract. When strengthening existing I-girder composite bridges one idea is to make the cross section act like a box section, by adding a horizontal truss between the bottom flanges. This means that eccentric loads produce a torque that is transferred by shear forces around the section. The magnitude of the effects coming from introducing such a framework between girders is addressed in this article. The fatigue resistance will be improved by the reduced stress ranges and increased amount of tolerated load cycles and extend the lifetime of the details, and by so the lifetime for the bridge. The work described in the paper is part of the European R&D project Prolonging Life Time of Old Steel and Steel-Concrete Bridges (ProLife), RFCS 2015-00025.

1. Introduction

Many old I-girder steel/composite bridges are too weak and in a need of replacement, repair or strengthening.

The rules for assessment/classification vary between the European countries. In addition, newer bridges designed according to the Eurocodes for example often give much tougher design in fatigue, which for example can give twice as large bottom flange in mid span than the old Swedish codes, which means that very few of the old bridges would survive a check with the Eurocodes for new bridges.

For symmetric I-girder bridges the loads from the weight of the steel and concrete are generally evenly distributed between the girders, just as for box girder bridges. For bridges consisting of two I-girders the concrete deck is often considered as simply supported in the transverse direction on top of the girders, meaning that a concentrated load on top of one girder will be distributed to only that girder, with no help from the second girder. In reality the torsional stiffness of the deck and the warping stiffness of the whole composite section however transfer some of the load so the real distribution can be about 90 % for the loaded girder and 10 % for the other, depending on the geometry of the bridge. For a box section this is not the case, the both halves parts of the box share the eccentric loads almost equally [1].

When strengthening existing I-girder composite bridges, one concept is to make the cross section act like a box section, by adding a horizontal truss between the bottom flanges. This means that the eccentric loads produce a torque that will be carried by shear forces around the section. The preferred type of truss is a K-truss, since other types will force the diagonals to take part in the global bending, which will make them sensible to buckling between the joints. This means a lot in the Ultimate Limit State (ULS), but even more in the Fatigue Limit State (FLS). In the FLS the fatigue is determined by the stress ranges in certain parts of the structure, for instance the welded details of an I-girder. If the girders can act together, sharing the moment from an eccentric load evenly should a lower stress range can be achieved. The increased amount of load cycles that the bridge can withstand, with the new distribution between the girders (70/30) can be up to six times, compared to an un-strengthened bridge with the distribution (100/0) [2].

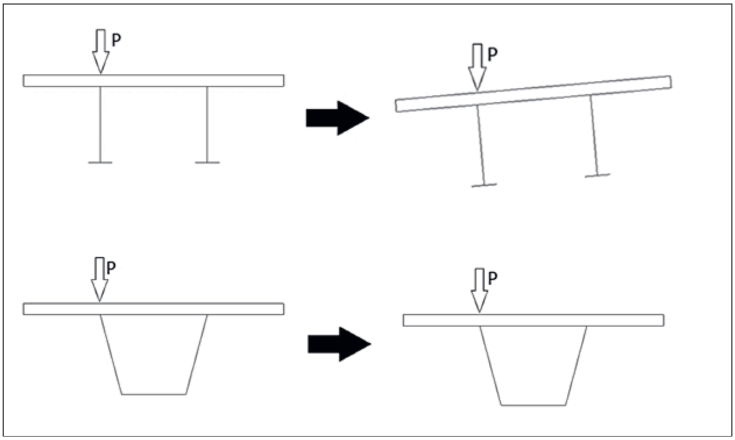


Figure 1. Deflection corresponding to different type of cross section.

2. Case study of the Pitsund Bridge

The Pitsund Bridge is a seven span bridge, with a total length of 399 m and a free width of 9 m, located in the northern part of Sweden outside the city of Piteå. In six of seven spans, the superstructure consists of steel girders with a concrete deck slab on top. The seventh span is a movable span, designed as a bascular bridge with two leafs.

This bridge was chosen for the case study with the strengthening method of a horizontal truss. The bridge has three different types of cross-section compositions. The first part which was strengthened in 2006 with coiled spring pins to achieve composite action. The second part which was original designed as a composite cross section with welded shear headed studs in 1984. The last span is a non-composite section with steel I-girders and a concrete deck with only a few anchoring rebars that connect the steel and the concrete.

The part chosen for this case study is the last one, indicated in red in Figure 2. This span is today like said without any composite action between the girders and the deck. The idea is that if a strengthening should be needed for this part of the bridge, the first part of the strengthening should be with the same method used in the first span, with the coiled sprig pins. There after the bridge could be strengthened further with a horizontal truss. The composite action provided by the coiled spring pins is essential for the cross section with the horizontal truss to work as a box section. This is because of that the non-composite cross section should be useless to be strengthened by a horizontal truss. Without composite action between the steel and the concrete the cross-section would theoretically still be an open cross section instead of a closed section which is the whole idea with the truss between the bottom flanges. In this case study is therefore assumed that this part of the bridge is a composite cross section.

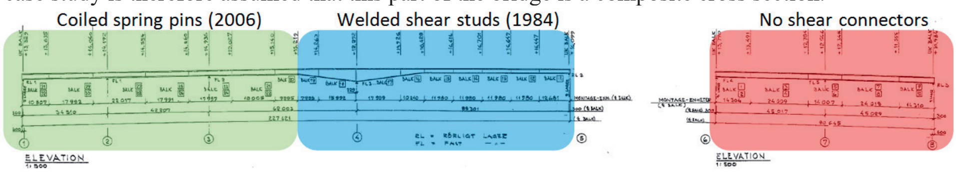


Figure 2. Elevation drawing of the Pitsund Bridge.

This part of the bridge is a continues bridge in two spans both 45 m long with a total length of 90.6 m including 0.3 m extension of the girders in both ends. The elevation drawing of the steel girders in these spans is shown in Figure 3.

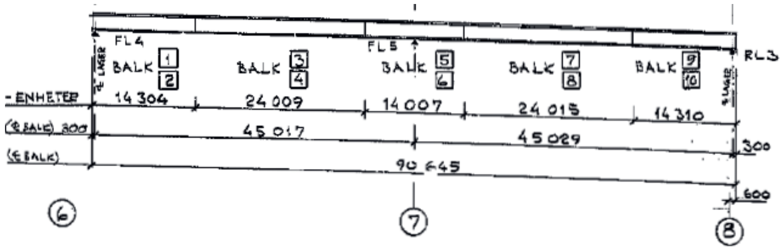


Figure 3. Elevation drawing of the chosen part for the case study**.**

The cross-section consists of two steel I-girders with a non-composite deck slab on top. Figure 4 illustrates the typical steel cross-section for this part of the bridge. The distance between the centre lines of the webs is 4.5 m, and the height of the steel section is ~2.4 m.

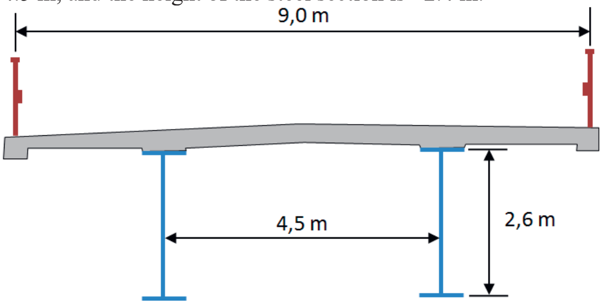


Figure 4. Typical cross-section of the superstructure for the chosen part of the bridge.

2.1. FEM-modelling

The FE-model and the analysis are made in the program ANSYS and the model consists of a concrete slab which is modelled by 8-node solid elements. The steel girders are modelled by 4-node shell elements and the cross-beams and horizontal trusses are modelled by beam elements. In Figure 5 the model is showed with different colours for the steel girders illustrating the different dimensions in the cross sections.

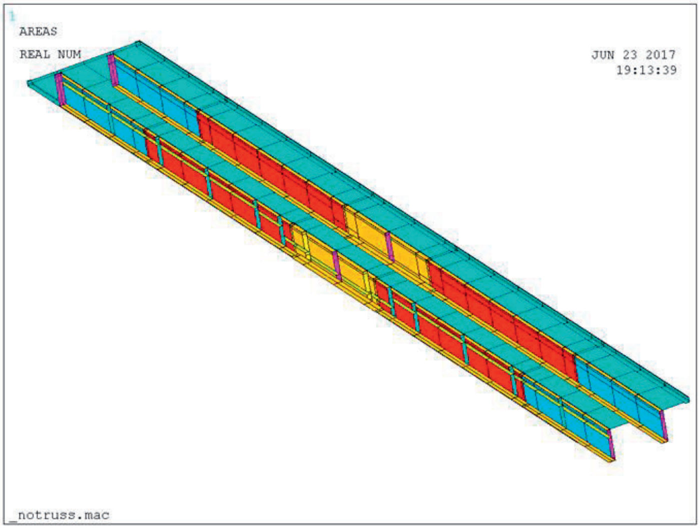


Figure 5. Bridge from below, no cross beams are showed.

The meshing of the model and the configuration of the cross beams are illustrated in Figure 6 and Figure 7.

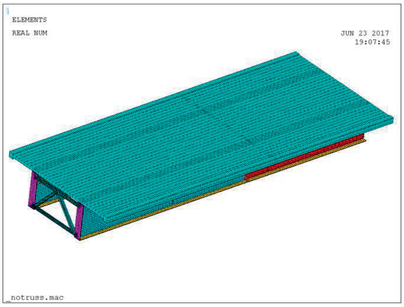


Figure 6. Bridge segment of the bridge from the above, no horizontal truss is showed.

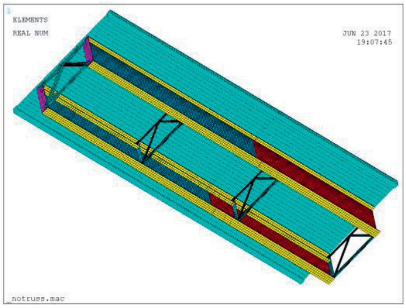


Figure 7. Bridge segment of the bridge from the below, no horizontal truss is showed.

3. Strengthening with a horizontal truss

Three types of shapes for the horizontal truss between the lower flanges are analysed in this study.

- K-shaped horizontal truss, see Figure 8
- X-shaped horizontal truss, see Figure 9
- D-truss, the truss is with diagonal beams between the lower flanges, see Figure 10.

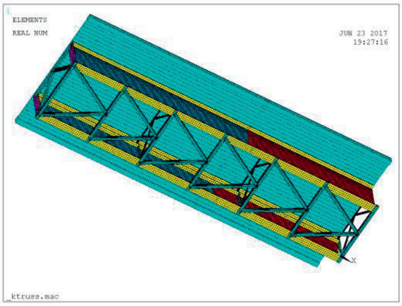


Figure 8. Bridge segment, from beneath, horizontal K-truss, RHS 200 x 200 x 10

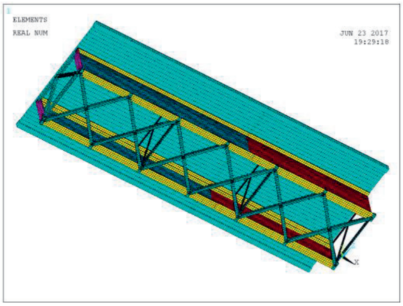


Figure 9. Bridge segment, from beneath, horizontal X-truss, RHS 200 x 200 x 10.

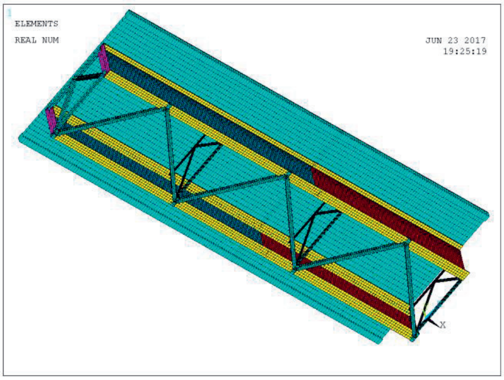


Figure 10. Bridge segment, from beneath, horizontal D-truss, RHS 200 x 200 x 10

4. Fatigue load

The fatigue vehicle used in this case study is the fatigue load model 4 [3]. For this study however the main aim is not to validate the fatigue damage but how much the stress from the fatigue load can be decreased by the strengthening method with horizontal truss.

Two main fatigue details are checked for unstrengthen and strengthened models. The first, located 66 m from the left (support 6), is the web stiffener, see Figure 11 and Figure 12. The other fatigue detail is the on-site welded joint, see Figure 11 and Figure 12, which is located at 76 m from support 6.

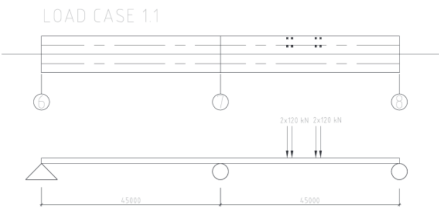


Figure 11. Load case for maximum stress at point 66 m from support 6.

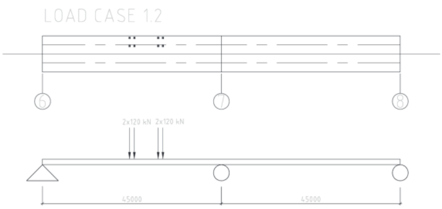


Figure 12. Load case for minimum stress at point 66 m from support 6.

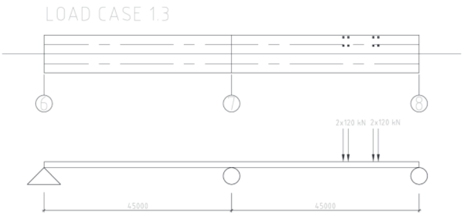


Figure 13. Load case for maximum stress at point 76 m from support 6.

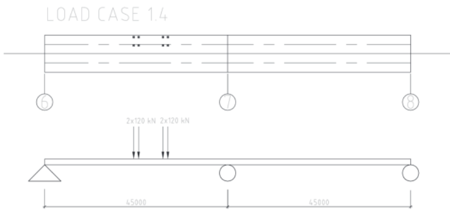
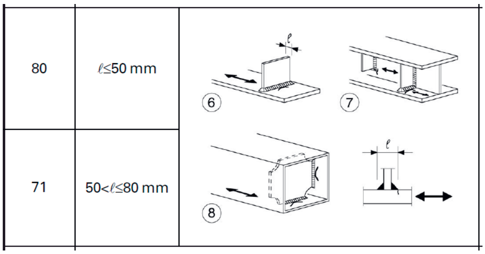


Figure 14. Load case for minimum stress at point 76 m from support 6.

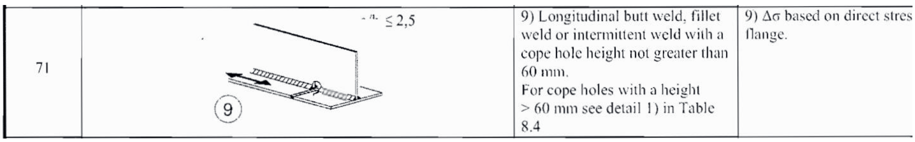
Figure 16 illustrates the fatigue detail of transverse splice, used for on-site welded joints, EN 1993-1-9, Table 8.2.

To be able to get the maximum stress amplitude from the fatigue load two cases for each fatigue detail must be tested. The first load case gives the maximum stress and the other the minimum stress. The loads where placed according to the influence line to get the highest respectively the lowest stress in the detail. The vehicle load was placed as far out as possible, which means 0.5 m from the edge of the traffic lane, wheels furthest out was 4 m from the center line of the bridge.



80	$\ell \leq 50$ mm	6, 7
71	$50 < \ell \leq 80$ mm	8

Figure 15. Detail classes according to EN 1993-1-9, for web stiffeners.



71	≤ 2,5 9	9) Longitudinal butt weld, fillet weld or intermittent weld with a cope hole height not greater than 60 mm. For cope holes with a height > 60 mm see detail 1) in Table 8.4	9) Δσ based on direct stres flange.

Figure 16. Fatigue detail of transverse splice, used for on-site welded joints, EN 1993-1-9, Table 8.2.

5. Results for the case study

The results from Pitsund Bridge are summarized below. The stress range for the location at x=66 m where the fatigue detail are located are listed in following tables. The result for the other location x= 76 m is not presented further in this article due to that it shows the same type of results. Some short explanation for the indexes follows:

$\Delta\sigma^1_m$ - stress range, in case of membrane stresses, in the most loaded girder

$\Delta\sigma^2_m$ - stress range, in case of membrane stresses, in the girder which not has the load directly above

$\Delta\sigma^1_{m+b}$ - stress range, in case of membrane stresses + lateral bending stress, in the most loaded girder

$\Delta\sigma^2_{m+b}$ - stress range, in case of membrane stresses + lateral bending stress, in the girder which not has the load directly above

The analysis is based on cracked concrete above the internal support, which is the design case for new bridges. The assumption is that the length of the cracked concrete is 15 % of the span length on both sides of the internal support; the assumption is according to the norms in EN.

Table. 1 Stress ranges bottom flange at x=66 m based on cracked concrete above internal support, loads at lateral coordinate z = 4 & 2 m except for No truss, centric load for which z = 1 & -1 m. Beam 1 is the most loaded beam.

Analysis	$\Delta\sigma^1_m$ [MPa]	$\Delta\sigma^1_{m+b}$ [MPa]	$\Delta\sigma^2_m$ [MPa]	$\Delta\sigma^2_{m+b}$ [MPa]	$\Delta\sigma^1_m + \Delta\sigma^2_m$ [MPa]
D 100 x 100 x 5	34.8	38.5	14.0	15.6	48.8
D 200 x 200 x 10	30.0	32.5	18.6	19.0	48.6
K 100 x 100 x 5	35.1	39.6	14.2	19.3	49.3
K 200 x 200 x 10	30.2	32.8	19.0	21.9	49.2
X 100 x 100 x 5	32.3	33.1	16.9	23.9	49,2
X 200 x 200 x 10	28.7	30.6	20.6	25.6	49.3
No truss	45.4	49.8	3.8	8.3	49.2
No truss, centric load	24.7	24.8	24.7	24.8	49.4

5.1. Improvement of stress ranges and estimated life time with respect to fatigue

The stress ranges for the strengthened models are here compared with the model without trusses.

Table. 2 Ratio of membrane + lateral bending stress for truss to no truss, most loaded beam, cracked above internal support, x =66 m

Analysis	$\Delta\sigma^1_{m+b}$ [MPa]	Ratio
D 100 x 100 x 5	38.5	0.77
D 200 x 200 x 10	32.5	0.65
K 100 x 100 x 5	39.6	0.80
K 200 x 200 x 10	32.8	0.66
X 100 x 100 x 5	33.1	0.66
X 200 x 200 x 10	30.6	0.61
No truss	49.8	-

6. Conclusions

The main purpose of the work carried out in this report has been to further analyse the effect from a horizontal truss system that has been added between the bottom flanges of the main I-shaped girders in a composite bridge. The main idea of applying these extra bracing members is that the connection of the two main girders will change the overall behaviour of the bridges from more or less two independent girders to a "box girder bridge" meaning that an eccentric load can be carried by a combination of bending and torsion in the box girder rather than "pure bending" in the loaded girder only.

Based on the results of the analyses of the Pitsund Bridge, the following results are found:

- The increased life-time for the fatigue detail in x=66m (web stiffener) on Pitsund Bridge various between 2.5 to 12 times depending on the configuration and amount of the horizontal truss.
- All three types of horizontal trusses are almost equalled efficient for purpose of distribute the eccentric load between both girders.
- Without any additional web stiffeners at the locations for the connection of the bracing the global stiffness contribution is limited, even for X-bracings which would have given a decreased stress in total for the global bending.
- Important note, to have any use of this horizontal truss the part of an old bridge without composite action it would first be needed to be strengthened with post installed shear connectors, for example with welded shear connectors or coiled spring pins.

Acknowledgements
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Paper III

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Torsion of a Norwegian bridge with partial box-action – a case study

Torsion of a Norwegian bridge with partial box-action - a case study

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Abstract

Some old bridges have a truss between the bottom flanges not intended for torsional effects but for transferring horizontal forces. This paper describes the effects of the truss on torsion for a Norwegian three span bridge from 1967, without composite action. Furthermore, the effects of post-installed shear connectors are investigated.

For composite bridges without intended composite action in bending, the effects of the slab preventing the top flanges from moving laterally should not be ignored, since this is important for the deformations of the girders under eccentric loading. Furthermore, the load distribution between the girders for an eccentric load is significantly enhanced if the horizontal truss is considered. The paper also investigates and presents the effects of post-installed shear connectors, with respect to bending stresses in the bottom flanges (moderate effects) and the top flange (large effects).

Keywords: Composite bridge, strengthening, horisontal bracing, composite action, box-action.

1 Introduction

The highest loads Norwegian roads are exposed to are mainly due to industrial timber harvest and special heavy transports. To increase redundancy and relax the most utilized routes, some of the roads are analysed and sometimes upgraded to accept higher load capacities to match the demand of the industry. Bridges being part of this network are often the bottlenecks of specific routes. The Norwegian Public Road Administration (NPRA) specifies requirements for classification of bridges for the specific uses. Classes are defined for bridges that have about 50 years or less remaining of the 100-year lifetime. The heaviest class for timber lorries allows for 60-tonne total load with 10-tonne axle load, referred to as Bk10/60, whereas requirements for special heavy transport falls into class Sv12/100 that requires the bridge to tolerate a 100-tonne total load with 12-tonne axle load. Load cases are defined more in detail in Handbook V412, [1].

Old bridges of different types are continuously assessed by bridge managers and the NPRA to update their classes. In the Norwegian road network, there are over 1700 steel girder bridges with a concrete deck without an intended composite action with the steel girders. The NPRA advises conservative assumption that the steel girders resist entirely the permanent and variable loads acting on the bridge. Many of these bridges can be reinforced by establishing a shear connection between the steel girder and the concrete deck to enable composite action.

Figure 1. Overview of bridge location, taken from north

2 The bridge in general

The Østre Trøsken bridge was built in 1967. The construction is a welded steel girder bridge with two identical girders in three spans and a concrete deck consisting partially of prefabricated elements. The total length of the bridge is 96m with the longest span being 51m for the center span, see Figure 5. The girders are aligned on top of the top flange and the cross-section varies in height and thickness with bolted splices. The bridge is slightly skew in the longitudinal direction and the girders are placed at slightly different heights. The difference in height is compensated by the concrete deck being built-up to result in a transversally horizontal deck.

The two girders are designed with horizontal cross bracings close to the bottom flange, see Figure 2, and vertical cross bracings at various locations including at the internal supports. These members are also bolted to the girders by means of bolted or welded stiffener plates. The horizontal bracing is designed with trusses in a X-shape. This means that the bracing could contribute to the global bending, due to its shape [2].

The bridge is highly utilized due to the long mid span and rather slender members. Several measures were considered including instatement of a pillar at midspan, but this presented significant environmental issues with regards to local diversity and river flow. For the new load picture, preliminary analyses showed the bridge has strength issues at midspan of the center span and over the internal supports.

The Handbook V412 [1] and V413 [3] require that the capacity of steel bridge components is verified according to NS 3472, September 2001 and does not allow the use of cross-section classes 1 and 2, meaning no plastic moment distribution is considered in the cross-sections. An evaluation of the strength by consulting company Norconsult

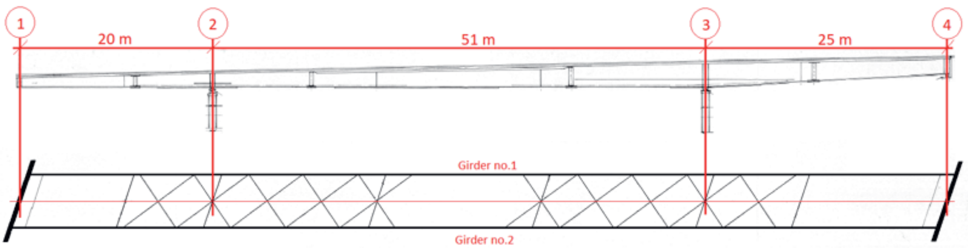

Figure 2. Elevation and configuration of the horizontal bracing.

shows the moment capacity is limited by plate buckling of the flanges at both critical locations. In addition, the cross-sections at the supports also present an issue with shear buckling of the web plates.

To solve the issue at midspan, the suggested strengthening consists of achieving composite action by welding headed shear studs on the center span over approximately 38m centered at midspan. Additional reinforcement of the bottom flanges is also required where composite action is achieved to fulfill the steel strength criteria. Composite action leads to a redistribution of the moment giving lower bending moments over the pillars. The difference in moment distribution is illustrated in Figure 3 for a uniform distributed load. Where the red moment distribution is for the design without composite action and blue is with composite action in the mid span. It is also proposed to reinforce the steel girders by bolting angle bars on the web, increasing moment and section buckling resistance. Shear strengthening has not been concluded yet.

3 Geometry and materials

The bridge deck is skewed at its ends with an angle around 21 degrees. The main dimensions and characteristics of the bridge are summarized in Table 1. The bridge has a spacing of 5.3 m between the I-girders, see Figure 3

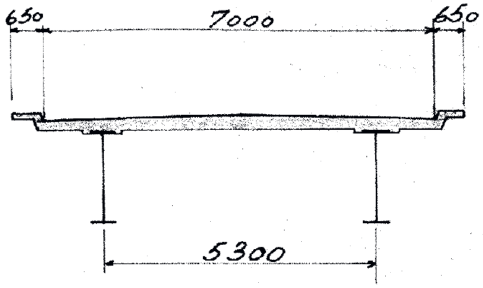

Figure 3. Bridge cross section.

Table 1. Information of the bridge

Year of build	1967
AADT*	1595 with 9% of heavy vehicles
Length of spans	20 m – 51 m – 25 m (west to east)
Free width (total with)	7.0 m; (8.3 m)
Height of steel girders	0.88 m to 2.30 m
Yield strength, f_y	355 MPa (OX 522D)**
Compressive strength of concrete slab	16.8 MPa
Thickness of concrete slab	220 mm to 265 mm

* Average annual daily traffic, AADT

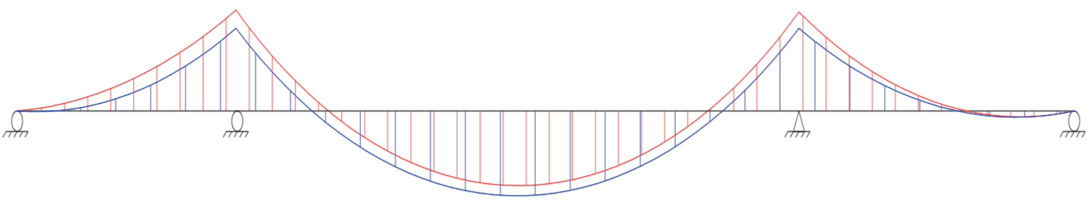

Figure 2. Moment distribution with two cases. Red: without composite action in mid-span, Blue: Composite action in mid-span.

** Steel from Oxelösund

4 Properties and modelling

The proposed solution for strengthening the bridge consist of achieving composite action and adding additional steel to increase the bending resistance of the cross section. The different parts of the cross sections are furthermore investigated and evaluated by its effect on both the structural behaviour of the bridge and the capacity of the cross section. The bridge is modelled and analysed for these four configurations:

1. Original design
2. Without the horizontal bracings
3. With composite action over the whole bridge
4. With composite action over the midspan, as chosen in the strengthening design proposal.

The effects from the different configurations are considered in the evaluation. These effects are, the vertical and horizontal displacement over the bridge length, the steel stresses in mid span on both girders and the max-/minimum normal force in the bracings are considered in the analysis.

The analysis is done with FE-models modelled in the FE-analysis program SOFiSTiK, with shell elements. The materials are taken from Table 1.

4.1 Cross sections

The bridge consists of two steel I-girders with additional plates on the flanges in mid- and support areas. On older plated steel girders additional plates were used to increase the flange area. These additional plates were used due to the limitation in rolling thickness from the steel plants in the Nordic countries at this time. In addition to the additional plates on the flanges, diagonal plates under the top flange have been used in the design, see Figure 5

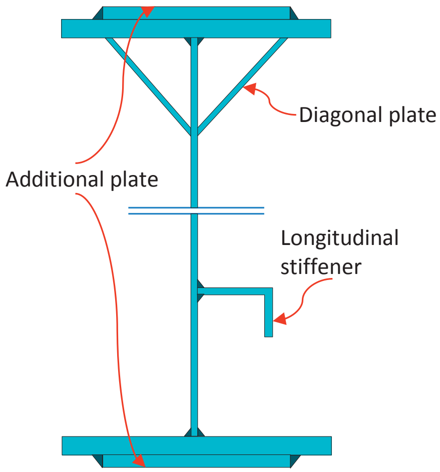

Figure 4. Explanation of the plates in the I-girders

The girders are also strengthened for buckling at support areas with longitudinal web stiffeners. Between the I-girders a horizontal bracing is fastened in the areas of the internal supports and out in the quarter area of the spans.

4.2 FE-modelling

As mentioned, the FE-program SOFiSTiK is used for the modelling and analysis of the different bridge configurations. The model is built up by shell element, beam elements (trusses and cross beams) and the original drawings have been used to identify including steel plates. Some simplifications have been done in the modelling design. The concrete area above the internal supports is designed as cracked by reducing the Elastic modulus so that the concrete area is equal to the longitudinal reinforcement. This is done over a length of 15 % of the spans. For this bridge a more exact estimation should be done because of the big ratio between the mid span and end spans. According to the EN the limit of the ratio between the spans is around 0.6 [4]. This means that this bridge should be analysed and where the extreme fibre tensile stresses in the concrete exceed twice the strength f_{ctm} or f_{lctm} the concrete should be modelled as cracked in the global analysis.

The beam joints plates and bolts are not either modelled. This has no impact on the result but in comparison of the dead load of the steel and the steel specification it is a difference of 2-3 %. The total amount of steel is around 84 tonnes.

The FE-model in SOFiSTiK includes as mentioned the whole structure, see Figure 5 and Figure 6, including the edge beams, which are assumed to be cracked along the whole length. The beam elements representing the cross beams and horizontal bracing are connected directly into the shell elements representing the web plates. The stiffness from the connection plates in the connection joint is there for disregarded, but that should only influence the results by a margin.

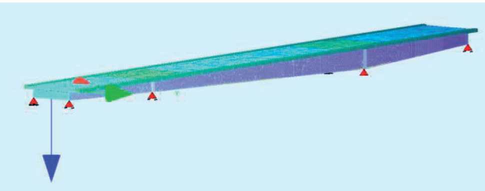

Figure 5. FE-model of the whole structure including the global coordinate system.

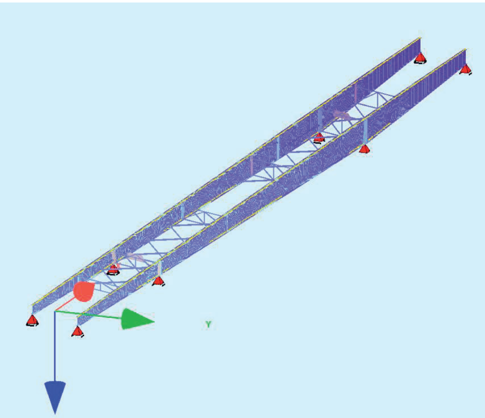

Figure 6. FE-model of the including steel elements, shells and beams.

The interface between the concrete deck and steel girders is modelled with line constrains between the two shell elements (centre of the web and to the centre of the concrete shell element). This constrain is modelled with stiffness in the corresponding horisontal directions for non-composite and composite action. However, for the non-composite action the stiffness in the transversal direction is modelled stiff. This because of that it is more to the reality, compared to the theoretical non-composite interface which should give no stiffness in both directions in the horizontal plane. In Figure 7 a schematic illustration of the connection between the interface of steel and concrete.

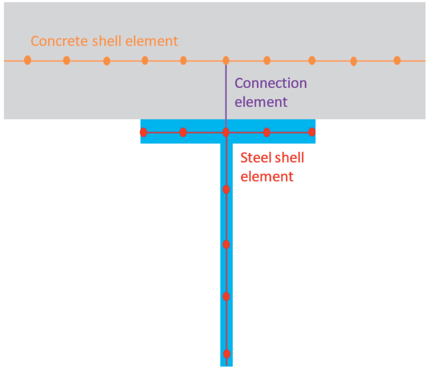

Figure 7. Illustration of the connection between the two elements (steel concrete).

4.3 Load for the analysis

To test the different configurations and to see the bridge behaviour an eccentric point load of 1 MN is used. The point load is placed on top of the centre of girder, indicated as girder no.1, G1. This load will give the relation of the load distribution between the girders, stress distribution and displacement of the girders in the mid-section and the normal forces which occur in the horizontal bracing due to the torsional moment in the section. The load is placed in the middle of girder no.1 which is not the centre of the bridge due to the slanted abutments and support lines. The values of the analysed results are however from the section at the centre of both girder no.1 and the girder no.2.

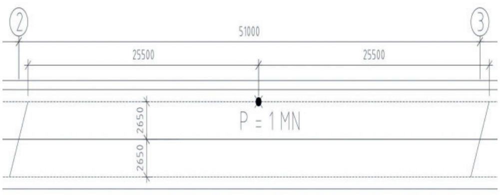

Figure 8. Placement of the point load on girder no.1.

5 Results

The results from the analysis of the four models of the different bridge configurations (M1-M5), are tabulated in Table 2 and Table 3. Results from a fifth configuration is shown in Table 3. M5 shows the results from a configuration of the original design but without any transversal composite action between the girders and the concrete deck. Explanation to the indexes in Table 2 and Table 3 can be found below with the corresponding units to the measured values.

σ_{top} – Nominal stress in top flange centre, [MPa]

σ_{bottom} – Nominal stress in bottom flange centre, [MPa]

$\delta_{y,top}$ – Out of plane displacement at top flange, [mm]

$\delta_{y,bottom}$ – Out of plane displacement at bottom flange, [mm]

δ_z – Vertical displacement, [mm]

N_{max} – Maximum normal force in truss, [kN]

N_{min} – Minimum normal force in truss, [kN]

Table 2. Results for configuration 1-2

	M1		M2	
	G1	G2	G1	G2
σ_{upper}	-231	-43	-233	-41
σ_{lower}	156	62	172	46
$\delta_{y,upper}$	-19	-19	3	3
$\delta_{y,lower}$	2	-1	40	40
δ_z	147	60	162	46
N_{max}	148		-	
N_{min}	-126		-	

Table 3. Results for configuration 3-4 and 5

	M3		M4		M5	
	G1	G2	G1	G2	G1	G2
σ_{upper}	-33	-2	-50	1	-225	-46
σ_{lower}	125	41	145	51	167	49
$\delta_{y,upper}$	1	1	-15	-14	-44	-41
$\delta_{y,lower}$	5	3	1	4	-12	-14
δ_z	57	28	98	39	-149	-54
N_{max}	182		124		19	
N_{min}	-186		-120		-19	

The stress distribution in G1 from the analyses and with the five different configurations is summarised in Figure 9. The distribution is showed from the two stress values in the flanges, which means that the distribution is disregarding any out of plane stresses.

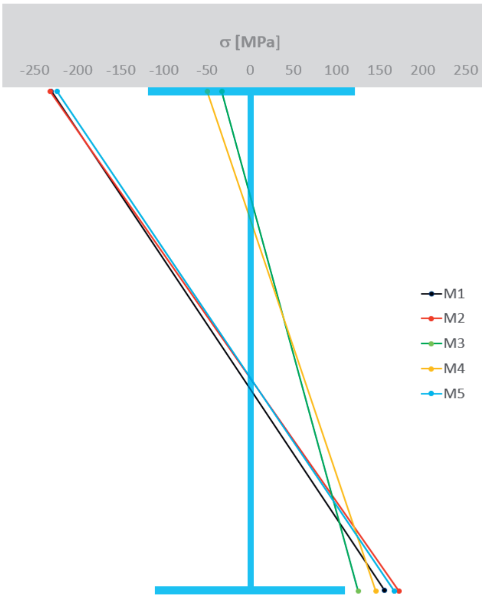

Figure 9. Stress distribution in G1 at the middle of the mid span from the results in M1-M5.

6 Conclusions

Some concluding remarks about the results from the comparison analysis are listed below. Some conclusions regarding strengthening principals of this bridge and strengthening in general for other composite bridges are also listed.

- There are many steel-concrete bridges which could gain from being strengthened from an economical point of view.

- The composite action in the transversal direction must be included. The horisontal stiffness in the transversal direction for the interface between the steel flange and the concrete deck are essential for the analysis. Without this stiffness the real behaviour of the bridge would not be captured.

- Achieve composite action in the mid span only can give enough increase of the bending resistance for the mid sections of the bridge, with respect to stresses in the top flange.

- The stress in the bottom flange will not decrease so much from achieved composite action of the cross section.

- All bridges are not designed to resist torsional moments. The wind bracings can make significant difference for a bridge like the Østre Trøsken bridge. The behaviour of this kind of bridge will nowadays be captured by any FE-modelling.

- The suitable strengthening methods differs a lot between bridges of this type and the strengthening at different sections for the same bridge. The strengthening technique for the support areas is not necessary the most beneficial method for the mid sections.

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Paper IV

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Strengthening of a Composite I-girder Bridge by Trusses Introducing Box-Action

Strengthening of a Composite I-girder Bridge by Trusses Introducing Box-Action

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Abstract. The increased amount of traffic and the increasingly heavier loads on the road network push the existing infrastructure to its design limit. Bridges, which are an important part of the road network, need to be adopted to the new traffic demands regarding both the load capacity and the fatigue limit state, FLS. Steel-concrete composite bridges, with twin steel I-girders, is a common bridge type in the Nordic countries. These bridges are often designed using beam models, assuming that the concrete deck is a statically determinate structure supported on the two steel girders in the transversal direction. This assumption implies that the most loaded girder can sometimes be subjected to even more than 100% of an eccentric load, e.g., the traffic loads in the design codes. If the girders are strengthened to be able to share the eccentric loads more equally, it would have a significant impact on the load capacity, especially for the fatigue limit state. By introducing horizontal trusses between the bottom flanges of the girders, making the cross-section act more like a box-girder, the torsional stiffness will increase so that the girders will share the eccentric loads more equally. The bracing system of the trusses can be designed in different shapes, each of them with pros and cons for the existing bridge structure. In this paper, the effects from different shapes of the bracing are evaluated of a single span I-girder composite bridge. The increased torsional stiffness and the change of the internal shear flow will increase the load capacity of the steel girders.

Keywords: bridge; steel-concrete composite; I-girder; case study; horizontal trusses; torsional stiffness; strengthening; composite bridges

1. Introduction

In the Nordic countries, a common bridge type is the twin steel I-girder bridge with a composite concrete deck slab. The cross-section of these bridges is not as torsional stiff as a corresponding box-girder, which has a closed cross-section. By installing a horizontal truss between the bottom flanges, a semi-box cross-section can be achieved. This cross-section will gain some of the benefits from a closed cross-section, as in a composite steel-concrete box-girder, with an increased torsional stiffness, see Fig. 1.

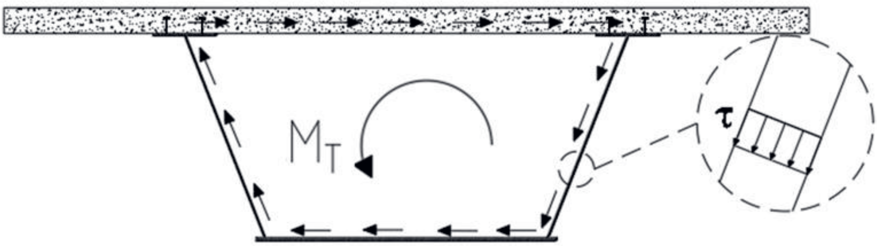

Fig. 1 Shear flow for a closed cross-section in a steel-concrete composite box-girder.

The impact on the load distribution, from eccentric traffic loads, between the two I-girders have earlier been studied by Vestman et al. (2018) and Ivanov et al. (2020). The result in these studies indicate that the implementation of a horizontal truss between the bottom flanges increases the load distribution between the girders, which lowers the bending stresses in the most loaded girder. The decreasing of the bending stresses from traffic loads, will in terms of bending moment- and shear capacity, be increased for the steel girders.

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On the other hand, for this type of bridges the load effects from the dead loads are quite large. This means that the decreased stresses from the eccentric traffic loads will not have the same impact on the ultimate limit state, ULS, as the impact on the stresses in the fatigue limit state, FLS. Other studies about the use of horizontal bracing system in steel girder bridges shows the benefits of different truss shapes. In both Fan and Helwig (1999) and Rageh et. al. (2012) the effects of horizontal trusses between the upper flanges in box-girders during construction and casting, are shown. In both studies, the results indicate, that the shape of the bracing system in many cases is not the most important factor. The same conclusions were drawn in the studies by Vestman et al. (2018) and Ivanv et al. (2020). It can however have an impact on other effects than just the load distribution of eccentric loads for the global bending. This needs to be considered in the design if horizontal trusses are used, e.g., if the new load distribution will increase the normal forces in the cross-frame members.

This study includes further numerical investigations regarding the use of post-installed bracing systems between the lower flanges in twin I-girder composite bridges. In this study the authors considered additional shapes of the trusses, compared to earlier studies. Also, the impact on existing structural parts as shear studs, support- and internal cross-frames are investigated. The Yxlö Bridge in Sweden was chosen as a case study since the bridge has been used in another study regarding post-installation of shear connectors. The composite behavior between the concrete slab and the steel girders is essential for the horizontal trusses to be able to form a semi-box girder with some of the benefits of a closed cross-section. In Tjernberg (2022) the effects from post installed shear connectors were theoretically investigated for the Yxlö Bridge. The results showed a substantial increase of the load capacity of the steel girders, by creating composite action with post-installed shear connectors.

2. The Yxlö Bridge

The Yxlö Bridge in Sweden is a simply supported bridge in one span of 31 meters over the Yxlö channel south of Stockholm, see Fig. 2. The bridge was built in 1961 and consist of two steel I-girders with a concrete deck on top of the girders. The bridge has two traffic lanes, one in each direction, and a free width just over 7 meters. The bridge is designed as a non-composite bridge which means that shear connectors was not designed to transfer a shear flow between the concrete and the top flange of the girders. Only a small number of steel-stirrups were used to prevent separation of the girders and the deck. This type of bridges, where the concrete is not designed to be in composite action with the steel girders was common up to the 1980’s in Sweden.

Fig. 2 The Yxlö Bridge (The Swedish Transport Administration, n.d.)

The detailed geometry of the bridge and its cross sections are presented in Fig. 3, Fig. 4 and Table 1. The cross-section used in the FE-analyse is an equivalent cross-section where the concrete deck is horizontal instead of the with the 5 % superelevation as the real bridge section has. This has only a minor impact on the result, and it is neglectable for the purpose of showing the effects of different truss-shapes. In Fig. 3 the two girders are denoted as girder 1 and 2. The bridge has cross bracings over the supports and two internal cross bracings located 9,8 meters from the supports.

The concrete deck slab has a thickness of 233 mm between the girders and decreases to around 150 mm at the edge beams. The concrete quality of the slab is K35 (~C35/45) with a characteristic compressive strength, f_{ck}, of 35,5 MPa and a modulus of elasticity, E_{cm}, of 34 GPa.

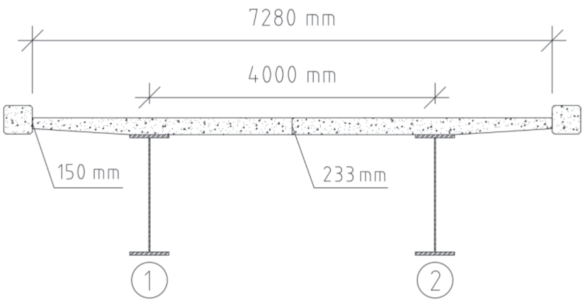

Fig. 3 The equivalent cross-section of the bridge used in the analysis

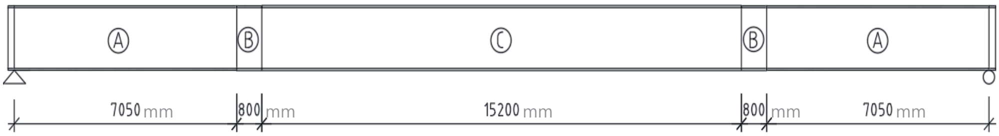

Fig. 4 Elevation of the steel girders

Table 1. Steel girder cross-section properties

Steel sections		A	B	C
Top flange	Width [mm]	550	550	550
	Thickness [mm]	40	40	55
Web	Height [mm]	1600	1600	1600
	Thickness [mm]	15	15	15
Bottom flange	Width [mm]	550	550	550
	Thickness [mm]	40	55	55

Note: The Youngs modulus of the steel, E_s, is assumed to be 210 GPa.

3. Finite Element Model of the Yxlö Bridge

In this study the FE-models are based on shell elements for the steel girders and the concrete slab, and beam elements for the cross bracings and horizontal trusses, see Fig. 5.

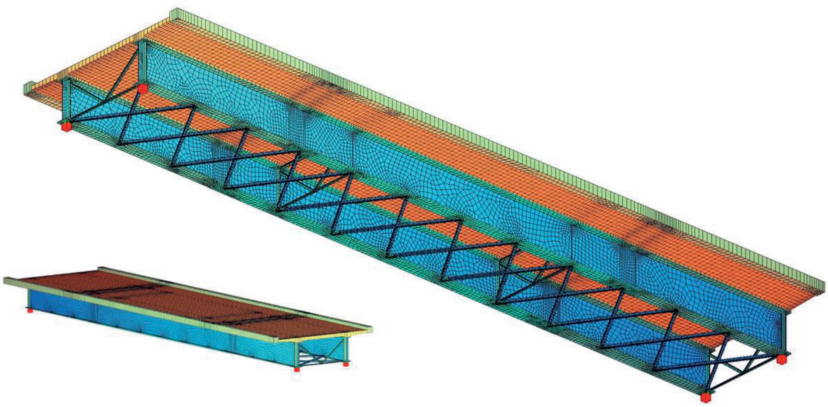

Fig. 5 FE-model showing the horizontal truss with the shape: X-truss

The steel-concrete interface is modelled with rigid connection elements to capture the theoretical behavior of a composite section, see Fig. 6. The commercial FE-software SOFiSTiK was used for the analysis. No consideration regarding the influence of the substructure has been taken since it has no significant impact on the results from vertical loads on a simply supported structure. The superstructure is thus modelled with nodal supports at the position of the bearings. The arrangement of the bearing system is with one fixed-, one unidirectional- and the other two as multidirectional bearings. This for the system to be determinate.

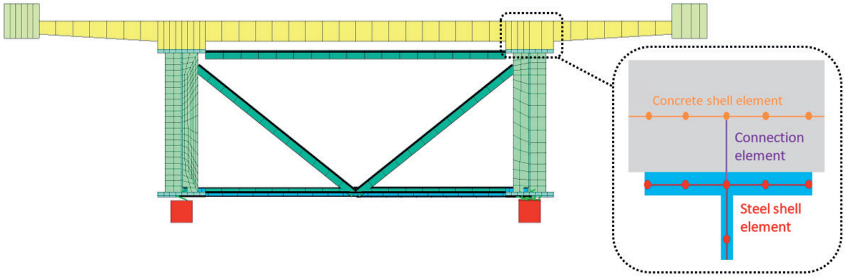

Fig. 6 Bridge cross section with a schematic illustration of the steel-concrete interface

The horizontal trusses were modelled with five different shapes and one additional alternative where only horizontal struts between the flanges were used. A squared hollow section, SHS, with the dimensions 100x100x4 mm was used for the horizontal trusses in all models. All six shapes are presented in Fig. 7 along with the original design of the cross-bracing without horizontal trusses. The different models are denoted follows, counted from top left in the figure: No truss (original), Horizontal, K-truss, X-truss, D-truss + horizontal, D-truss and Z-truss.

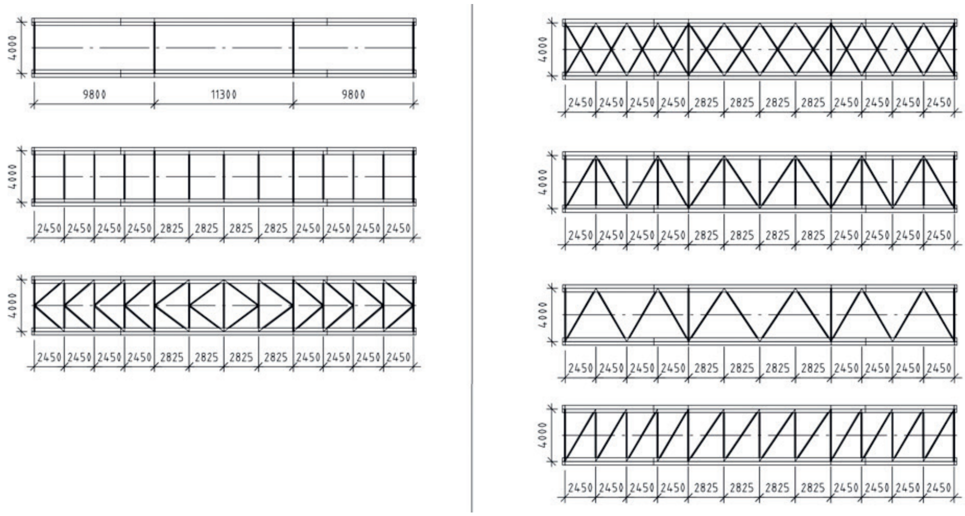

Fig. 7 Layout and shapes of the original design and the six different bracing systems

Two load cases were used in this study: one load case with an eccentrically located load and one load case with the load placed symmetrically at the middle of the deck. The two load cases will be called centric and eccentric. The analyzed load is the Fatigue Load Model 3, FLM3 from the Eurocode, EN 1991-2 (2005). The axle loads for FLM3 are 120 kN. The eccentricity from the center line of the bridge to the center of the axles for load cases and their longitudinal position is shown in Fig. 8.

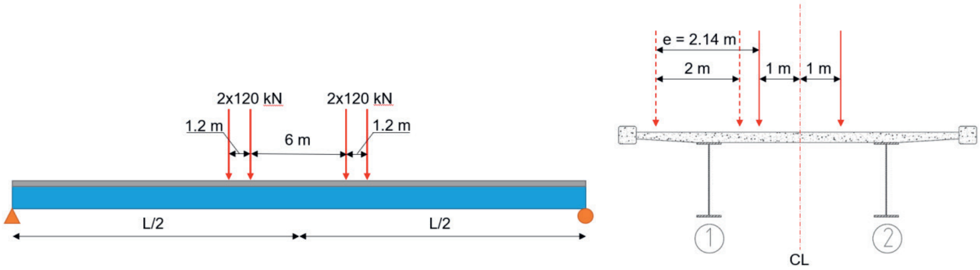

Fig. 8 Load position for the two load cases

4. Results

The deflections and the membrane stresses (σ_m) and the lateral bending stresses(σ_b) from the two load cases and the seven models are presented in Table 2 and Table 3. The deflection is taken at the middle of girder 1 and 2 and the stresses are taken in the middle of the bottom flanges at the same location of the girders. In Table 2 the results for both vertical-, δ_v, and transversal-, δ_t, deflection is presented for both girders. The sum of the vertical deflection of the two girders, $\delta_{1,v} + \delta_{2,v}$ is also presented. In Table 3 the corresponding values for eccentric load case, as for centric load case in Table 2, are presented

Table 2. Deflection at mid sections for load case: centric

Type	$\delta_{1,v}$ [mm]	$\delta_{1,t}$ [mm]	$\delta_{2,v}$ [mm]	$\delta_{2,t}$ [mm]	$\delta_{1,v}+ \delta_{2,v}$ [mm]
No truss	7,0	0,1	7,0	0,1	14,0
Horizontals	7,0	0,0	7,0	0,0	14,0
K-truss	7,0	0,0	7,0	0,0	14,0
X-truss	7,0	0,1	7,0	-0,1	14,0
D-truss + horizontals	7,0	0,0	7,0	0,0	14,0
D-truss	7,0	0,1	7,0	0,1	14,0
Z-truss	7,0	0,0	7,0	0,0	13,9

Table 3. Deflection and stress at mid sections for load case: eccentric

Type	$\delta_{1,v}$ [mm]	$\delta_{1,t}$ [mm]	$\delta_{2,v}$ [mm]	$\delta_{2,t}$ [mm]	$\delta_{1,v}+ \delta_{2,v}$ [mm]	$\sigma_{1,m}$ [MPa]	$\sigma_{1,m+b}$ [MPa]	$\sigma_{2,m}$ [MPa]	$\sigma_{2,m+b}$ [MPa]	$\sigma_{1,m} + \sigma_{2,m}$ [MPa]
No truss	12,8	6,1	1,2	6,1	14,0	37,0	40,0	3,4	6,3	40,3
Horizontals	12,8	6,1	1,2	6,1	14,0	37,3	40,6	3,4	6,3	40,7
K-truss	10,4	2,3	3,7	2,3	14,0	29,5	29,6	10,6	10,6	40,0
X-truss	9,9	1,4	4,1	1,2	14,1	28,7	29,4	11,8	12,6	40,5
D-truss + horizontals	10,8	2,6	3,3	2,5	14,1	30,8	32,2	10,5	12,0	41,3
D-truss	10,8	2,6	3,3	2,5	14,0	31,0	31,4	9,4	11,4	40,4
Z-truss	11,1	3,1	2,9	3,1	14,0	32,1	32,3	8,4	8,8	40,5

The normal forces in the members of the cross bracing located x= 9,8 meters from the support are presented in Table 4. In Fig. 9 the cross bracing is illustrated with the index of each bar member and the arrows indicates the positive direction of the normal force in each member.

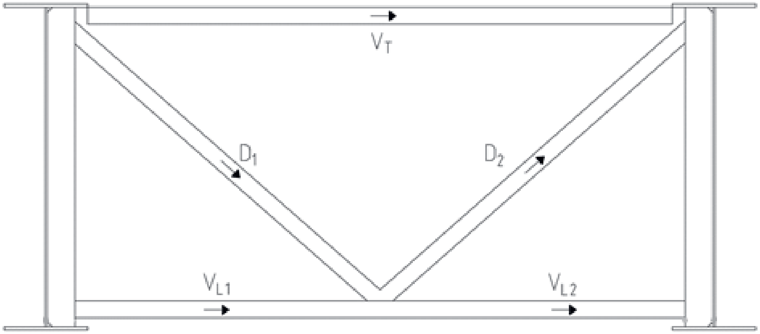

Fig. 9 Internal cross-frame with numbered bar-member

Table 4 Normal forces in the internal cross-frame at x = 9,8 m for the load case: eccentric

Type	D_1 [kN]	D_2 [kN]	$V_{L,1}$ [kN]	$V_{L,2}$ [kN]	V_T [kN]
No truss	-2	2	2	-2	-5
Horizontals	-2	2	2	-2	-5
K-truss	-42	42	37	-39	-5
X-truss	-48	48	31	-43	-5
D-truss + horizontals	-35	35	15	-40	-6
D-truss	-36	36	15	-40	-6
Z-truss	-28	28	-7	-51	-6

The shear flow from the eccentric load at the steel concrete interface of girder 1 is presented in Fig. 10 for each of the seven models. Also presented in the figure is the longitudinal load position of the axles. To clarify the difference between the model cases a segment, at x-coordinate 29-30 meters, of the shear flow diagram in Fig. 10 is shown in Fig. 11.

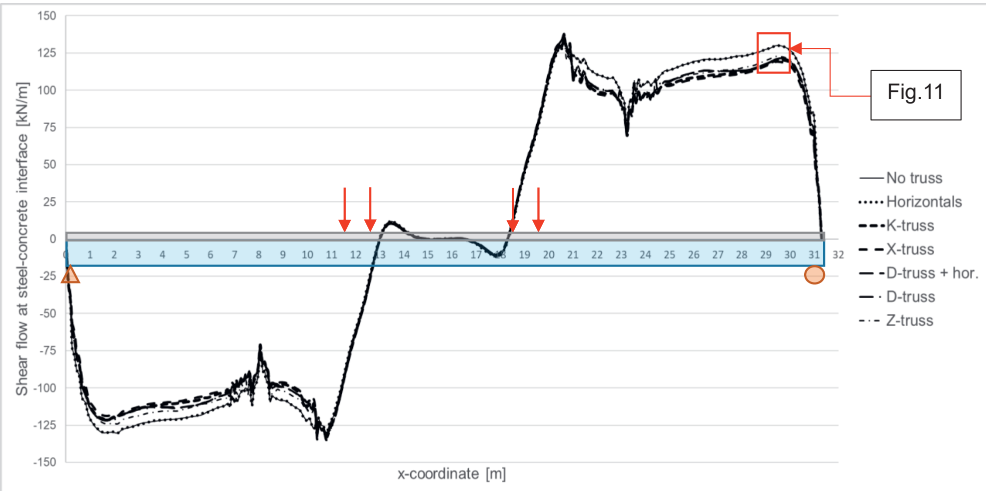

Fig. 10 Shear flow at steel-concrete interface along girder 1 for the eccentric load case

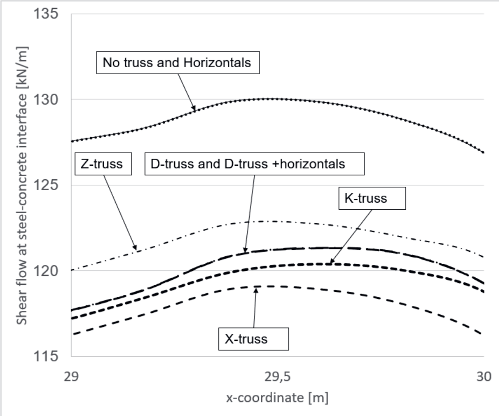

Fig. 11 Detail view of Fig. 10

5. Analysis

As the results from the centric load case show, the bridge models behave symmetrical. The deflections presented in Table 3 indicates that no truss shape, with SHS 100x100x4 mm, has significantly increased the total bending stiffness of the cross section. The X-truss would by its shape give some increase of the stiffness. In this case however, the lack of additional web stiffeners at the location of the truss-flange connection is limiting the contribution from the truss to the global stiffness of the cross-section. The total amount of deflection for the two girders is by that identical for all the cases.

The stresses in Table 3 indicates that the shapes of the trusses have some impact on the load distribution between the girders. The difference in the stress level, at the analyzed section, is however small between the truss-shapes. As for the deflections the sum of the stresses in girder 1 and 2 is almost identical between the analyzed models. If the load effects are represented by the proportion of deflection or stress in girder 1 compared to the sum of the deflections or stress in girder 1 and 2, the following table can be used to evaluate the impact from the different trusses.

Table 5 Comparison of the load effect distribution

Type	$\delta_{1,v}$ / ($\delta_{1,v}$+$\delta_{2,v}$) [-]	$\sigma_{1,m}$ / ($\sigma_{1,m}$+$\sigma_{2,m}$) [-}
No truss	0,91	0,92
Horizontals	0,91	0,92
K-truss	0,74	0,74
X-truss	0,71	0,71
D-truss + horizontals	0,77	0,75
D-truss	0,77	0,77
Z-truss	0,79	0,79

From the resulting normal forces in the cross-bracing it can be noticed that the implementation of a horizontal truss, except for the case with only horizontals, will change the shear flow and thus also the member forces in the existing structure. The load used in this analysis is used in design for fatigue verifications. Nevertheless, the increase of the normal forces in the cross-bracings, from 2 kN up to almost 50 kN for this load model, indicates that a verification of the existing structural elements is essential when implementing box-action in a twin I-girder composite bridge.

The impact on the shear force distribution can be evaluated from the resulting shear flow diagram, Fig. 10, which shows the shear flow at the steel-concrete interface. The detailed view in Fig. 11 indicate that the maximum decrease of the shear force, or the best distribution of the load between the girders, is with the horizontal x-truss. The shear flow between the steel and the concrete is decreased from 130 kN/m to 118 kN/m, a difference of 9%, which can be compared to the 22% difference when deflections or stresses are compared, see Table 5. This is expected, since even for a closed box girder with eccentric loading (where the deflections of both the left and right web are almost the same) the webs will have substantially different shear forces. This is since the webs will get an additional contribution of the total torsional moment from eccentric loading acting on the cross section.

6. Conclusions

The main purpose of the study has been to further analyze the impact of a horizontal bracing system on the existing structure. The bracing system consists of trusses between the bottom flanges and has theoretically been investigated on an existing I-girder composite bridge. By implementing horizontal trusses, the overall behavior of the structure will be changed to a box-action behavior instead of the twin girder system. The result from this study has confirmed a more equal load distribution between the girders. In addition to these results the reduction of shear force and additional normal forces in the internal cross bracings have been analyzed. The concept is however dependent on composite action between the steel girders and the concrete deck, which could also be obtained for existing bridges by e.g., the use of post-installed coiled sporing pins, see also Hällmark (2018) and Tjernberg (2022).

Based on the result from this study, regarding adding horizontal trusses on an existing twin I-girder composite bridge, the following conclusions can be drawn:

- Any shape of the studied horizontal trusses is approximately equally efficient in distributing an eccentric load between the girders, except the shape with only horizontal members.
- Both with and without trusses the ratios of deflections and stresses between the girders for eccentric loading in the two girders are similar.
- The impact from the horizontal truss on the shear force distribution is more limited than the impact on the bending stresses, due to the nature of the shear force distribution for a box-section.
- Additional checks are needed for internal members, like the cross bracings between the girders, when a horizontal truss is added. In this case an increase of around 45 kN (25 MPa).
- In contribution to post-installed shear connectors the horizontal truss could increase the load capacity or at least increase the fatigue limit for existing I-girder bridges with a composite concrete deck.

Acknowledgments

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Paper V

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Lateral trusses between I-girders introducing torsional stiffness to a composite bridge in Guatemala

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Lateral trusses between I-girders introducing torsional stiffness to a composite bridge in Guatemala

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Abstract

The ways of designing and building steel girder bridges with a composite concrete deck vary much between different parts of the world. A bridge system with twin steel I-girders or a concept with multiple, more than two, girders are used. The design and use of details and secondary systems also vary a lot. Commonly to give horizontal stabilization and to distribute horizontal loads bracing between the I-girders bracings are used. Although not so commonly, the bracing can also be used to distribute vertical loads between the main girders. This is possible for a steel-concrete composite bridge using a lateral bracing between the bottom flanges, which give a substantial St. Venant torsional stiffness to the cross section. For a twin steel I-girder composite bridge this leads to a box-girder behaviour, which leads to a better distribution from eccentric vertical loads between the girders. For a twin girder system, the principal is quite straightforward and not so many additional effects except the change from an open to a closed cross section need to be considered. Conversely, for a multiple girder system, which is statically indeterminate, additional effects need to be carefully investigated. To describe and to analyze these possible effects, this paper describes the design of a curved bridge in Guatemala City and its challenges. The new Bridge over the Pinula River is designed as a steel-concrete composite bridge with multiple steel girders with a concrete deck on top and has lateral bracing between the girders.

Keywords

Bridge, I-girder, lateral bracing, launching, steel-concrete composite, torsional stiffness

1 Introduction

A solution to design an economical bridge is to use the principal of a steel-concrete girder system. For this type of bridge, the two materials (concrete and steel) can effectively be optimized and use where they are most suited. The concrete is used as an overlaying slab carrying the live load which is distributed on the steel girders. The steel girders can be installed to its final positions and one way for the installation is to use incremental launching where the bridge is pushed or pulled. When the launching is finished the steel superstructure can be used as a part of the casting structure for the concrete [1]. So, in one meaning the steel superstructure is a permanent formwork. A steel-concrete composite bridge can easily be adapted for the bridge location and its environmental conditions. The different design of this type of bridges vary around the world. For instance, a system with two welded steel girder is more commonly in the Nordic countries than in the southern part of Europe where multiple rolled girders often are used [2]. In some countries like the U.S the twin I-girder system is recommended due to their way of considering the redundancy of the bridge superstructure. Independent on the tradition for the design and construction and the difference in the design code regulations in each country the steel-concrete composite bridges have some sort of concrete deck and two or more longitudinal steel girder (welded- or rolled section) [3]. The steel girders are also in some way connected by intermediate cross frames or diaphragms and in some cases with lateral bracings. The use of lateral bracings is however more widely different and is often used for stabilization of horizontal loads, like wind loads and is then called wind-bracings [1]. They could also be used as stabilization in the construction stage where the bracings are connected between the top flanges of the steel girders to secure the lateral stability of the system [1]. Not so widely used, or at least not to the authors knowledge, they can be used to distribute vertical loads in service limit state, especially eccentric vertical loads. The use of lateral bracing for construction stages is however well known both for I-girder- and box girder sections [4]. For a steel-composite bridge with two steel girders, lateral bracing would make the cross-section act more like a box girders section which has much larger torsional stiffness than the open section with the girders and the concrete deck [5]; [1]. The effects of lateral bracings have been studied for different type of composite bridges, but often to describe the bridge behaviour for the concrete casting stages and the distribution of the dead loads [6]. The distribution of the live loads seems not to be as important, and it could maybe be because of that dead loads and the loads from the construction stages in many cases have a large impact of the total design. Where it however could be important to distribute live loads and lowering the stresses in the steel girders is in the fatigue limit state which in many cases is one of the governing cases when designing steel members in bridges. A small reduction of the fatigue stress could make a huge difference in the design life for the bridge [7]. Also, to be able to increase the allowed load bearing capacity of an existing bridge a better distribution could mean that the bridge could continuous be used in service instead of being replaced with a new. This could save money for the bridge owner and postpone additional investments into the future [8]. To investigate the impact from lateral bracings on the load distribution a study of a new bridge in Guatemala has been done. In this study the existing bridge and its design has been used and compared with an additional use of lateral bracings.

2 Bridge description

The new Bridge over the Pinula River is part of the new south access road to Guatemala City called VAS ("Vía Alterna del Sur", which means "Alternative Southern Carriageway"). The bridge is necessary for the road to span the Pinula River in the municipality of Villa Canales.

The bridge is designed with as a steel-concrete composite bridge with welded steel I-girders and a concrete slab on top of the girders. It has an overall length of 161 m, divided into three unusually uneven spans, with lengths of 51+50+60 m. The bridge is curved in plan, with a curvature radius of 148 m and a constant longitudinal slope of 3.2%, uphill from abutment 1 to 2. The cross-section has a superelevation of 7.6% and the total deck width is 17.96 m, see Figure 1.

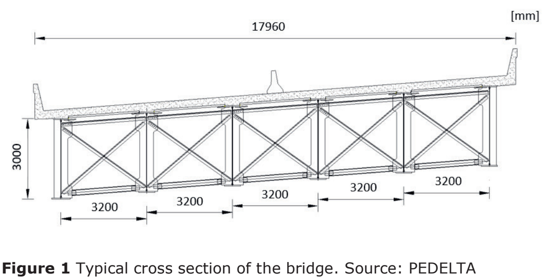

Figure 1 Typical cross section of the bridge. Source: PEDELTA

The superstructure consists of a composite deck of 3340 mm constant depth, composed of six welded steel I-girders of 3000 mm depth with a reinforced concrete slab of 275 mm thickness, slightly raised above the girders with concrete haunches to accommodate geometrical irregularities of the top of the steel beams from their tolerances during construction. The girders are evenly spaced with a centre distance of 3200 mm and the slab has cantilevers of 979 mm. Support diaphragms and intermediate cross-frames are provided between the longitudinal girders, as well as lateral bracing on top of the girders. The diaphragms and cross-frames are arranged in radial configuration in plan and with a spacing of approximately 3 m. The arrangement of the girders and the lateral bracings including the curvature of the bridge can be seen in Figure 2 which shows the bridge during the construction stage.

Figure 2 Overall view of the steel girders and bracings just after completing their construction. Source: PEDELTA

The bridge has been designed in accordance with "AASHTO LRFD Bridge Design Specifications", 8th Edition (2017) and "AGIES NSE 2" (2018), the structural local code from Guatemala. Other Codes as "AASHTO Guide Specifications for Seismic Isolation Design", 4th Edition (2014), "AASHTO Guide Specifications for LRFD Seismic Bridge Design", 2nd Edition (2009), and the Eurocodes for structural design have been considered additionally.

The bridge was designed for the HL-93 live load, even though other bridges in the country are designed for higher or lower loads, depending on its location (urban or close to heavy industry locations like mining facilities). The Peak Ground Acceleration at the bridge location is 0.53 g, which is moderately high even during construction.

The bridge was designed to fulfil infinite fatigue life (ASLDTT = 4669).

The compressive strength of concrete of the deck slab is 30 MPa, while the steel plates and profiles are ASTM A572 and ASTM A992 grade 50W respectively, with a tensile yield strength of 50 ksi (345 MPa), this is, equivalent to European Grade 355. All the joints, including the field joints, are bolted, with ASTM F3125 bolts, grade A490. All the bolted connections are designed as slip critical, except during seismic event.

Each I beam has different longitudinal grade and pre-camber which has strong impact in its design and the construction analysis as explained in chapter 3. The maximum pre-camber is 215 mm on the external girder and only 60 mm on the internal girder at the same longitudinal section.

2.1 Characteristics of the bracings

The bridge has lateral bracings between the top flanges of the six I-girders along the whole length. The bracings consist of L5x5x3/4" profiles and have bolted connections. A plan view over the arrangement of the I-girders and the top lateral bracings is shown in the top of Figure 3.

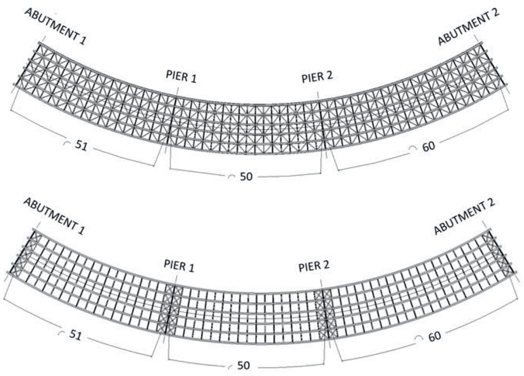

Figure 3 Top: Arrangement of the top lateral bracings. Bottom: Arrangement of the bottom lateral bracing.

The bottom flange of the I-girders is also connected with lateral bracings, but only between the intermediate cross frames and diaphragms at the four support regions. These bracings consist of a heavier profile, ½ WT 12x96 beam, than the top lateral bracings. The lower lateral bracings are designed to distribute seismic forces at the ends of the beams close to the support diaphragms. This to lower the in-plane bending stress at the lower flange due to the final seismic transverse shear, which otherwise would have been excessive. So, by adding these lateral bracings between the lower flanges at a limited area the lateral stresses from seismic horizontal bending from the supports were limited to acceptable levels. Theses bracings were installed after the launching stage to avoid undesirable effects. In Figure 4 the bolted connection for the bracings on the inner I-girders are shown

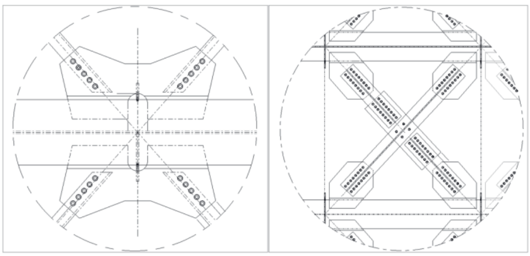

Figure 4 Left: Detail of the connection for the top lateral bracings. Right: Detail of the connection for the bottom lateral bracing.

3 Design and construction approach

For this bridge an incremental launching method was chosen due to the lack of accessibility for equipment and material to the riverbed (a seasonal river) and the availability of space at one of the abutments (future toll plaza). To make the launching possible a launching nose, see Figure 5, were attached at the front section of the steel structure.

Figure 5 View of the bridge during launching, including the launching nose. Source: PEDELTA

The strong in-plan curvature of the bridge and the necessary precamber of the steel girders cause certain deck areas to come off the rollers during several launching stages and distribute reactions on these temporary bearings and the deck itself, which made it challenging to estimate the reactions. To accurately obtain these reactions a three-dimensional nonlinear finite elements model was used. In this model, the longitudinal girders are "composite" type elements (frame elements for top and bottom flanges and shell elements for the webs), the cross-frames and bracings are frame type and the concrete slab is made of shell elements, see Figure 6.

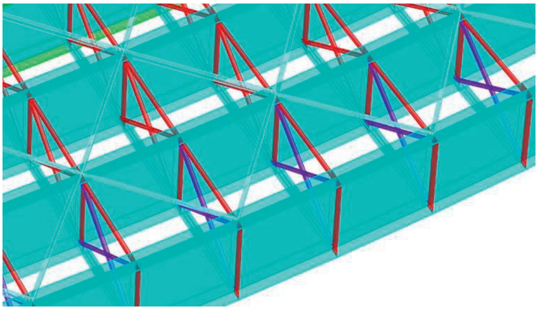

Figure 6 Detailed view of the 3D model, showing main girders, cross-frames, lateral bracing and web stiffeners. Source: PEDELTA

The commercial structural software used for the launching analysis was SAP-2000 (v20.2.0). In the model, following the classical approach, different radial axes of support successively activate and deactivate under the stationary bridge, thus providing different structural configurations for each stage of launching. Model bearings allow for the bridge freely uplift ("gap" type supports), which was expected to happen, due to the precambering of the steel girders.

4 Effects of the lower bracing

The bridge was designed without lower bracing at most of its length to provide flexibility during the launching stages. At these stages, a high torsional stiffness provided by the bracing would have had an undesirable effect. The very curved cantilever that is inevitable during launching imposed so large torsional forces on the bridge that the use of the lower bracing was impractical, regardless any possible benefit during launching. Moreover, the different precambering between the girders at each longitudinal section combined with a high torsional stiffness of the cross section would have made that only one or two girders were in contact over the temporary bearings during launching. Therefore, the girders, intermediate cross-frames, and temporary bearings, should have been heavier and more costly.

A soon as the construction stages govern the design of the girders, in this case and with this construction method and curved geometry, it made no sense to design the bridge with a high torsional stiffness even though it could be beneficial for the service stage once the bridge is opened to traffic.

Nevertheless, a comparison of the effects from installing lateral bracing between the bottom flanges in service conditions compared with the actual design with the lower lateral bracings only close to the bearing diaphragms. The lower bracing, used in the model for the comparison, was designed with angles L127x127x19,1 mm.

The software used for the comparison was SAP-2000. Design trucks geometry and location has been adapted to provide the maximum bending moment on the third span, which is critical along the bridge length. Joint loads are used for each wheel of the truck axes. Uniformly distributed lane loads, directly applied over the concrete slab shell elements of the model, have been also considered compatible with the trucks (HL-93 live load model). The design loads are adjusted with the AASHTO multiple presence factors, depending on how many lanes are loaded in each combination. The results are finally enveloped to provide the maximum outcome.

4.1 Results

In Table 1 the results for the maximum stress from the traffic loads in the exterior girder (girder 1) is presented together with the associated stress in the interior girder (girder 6).

Table 1 Comparison of stresses in the bottom flange, with/without lower bracing, as built, during service.

	With lower bracing		Without lower bracing	
Loads	Girder 1 (exterior)	Girder 6 (interior)	Girder 1 (exterior)	Girder 6 (interior)
Live load* – the two most exterior lanes loaded	27,1 MPa	5,1 MPa	37,6 MPa	0 MPa
Live load* – all lanes loaded	24,3 MPa	20,9 MPa	29,3 MPa	12,7 MPa
Total stress – all design loads	176,9 MPa	88,3 MPa	186,1 MPa	95,2 MPa

* Enveloped values for the worst scenario for the exterior girder

The same type of comparison of the impact from the lower bracing on the load-induced fatigue stresses were analysed. In Table 2 the stresses from the factored fatigue loads are presented. The stresses are enveloped with the maximum stress for the two girders with and without the additional lateral bracing between the bottom flanges.

Table 2 Comparison of stresses in the bottom flange, with/without lower bracing, as built, during fatigue load.

	With lower bracing		Without lower bracing	
Loads	Girder 1 (exterior)	Girder 6 (interior)	Girder 1 (exterior)	Girder 6 (interior)
Live load – fatigue load	17,1 MPa	25,5 MPa	23,2 MPa	31,9 MPa

4.2 Analysis

The additional lateral bracing between the bottom flanges reduces the live load stresses in the outer girders (exterior and interior) around 10 MPa, which is 5-7 % of the maximum total stress. This reduction is rather low and have a small impact on the design for this specific bridge. Also, the total stresses on the bridge in service are remarkably low, considering that the steel has a tensile yield strength of 345 MPa. This is however related to that the critical stage governing the design of the steel girders is the intermediate launching stage. The additional lateral bracings have almost a negligible impact on design of the steel girders.

As can be seen for the fatigue loads, the additional lateral bracing reduces the fatigue load stresses in the analyzed girders by about 6 MPa, which corresponds to about a 20-26 % reduction for the interior and exterior girder. This is a lot in the fatigue limit state, FLS. Although this is relevant in percentage terms, as in the previous exercise, the live load fatigue stresses on the bridge are far from the limit of a detail type "B", 110 MPa. Note that girder joints on-site were "combined" type: the bottom plates were welded, ground flush and smooth, to allow the launching, while the webs have a slip critical high strength bolted joint. So, again, due to the special features of the bridge the additional lateral bracing has a small impact on the design. However, this shows that lateral bracing between the bottom flanges will reduce the stresses from eccentric loads by increase the load distribution between the girders. This is true, but only if the top flanges of the girders are connected by bracings or with a composite concrete deck. This enable the cross-section to distribute the shear flow as a closed section, in this case as a multiple cell closed section.

5 Conclusions

The effects of a design with and without lower bracing have been compared in a recently built composite bridge. The bridge is strongly curved in plan and was built by the incremental launching method. The additional lower bracing would have reduced the live load stresses in the girders by 5-7% and the fatigue stresses by 20-26%. Therefore, it may have benefits during service stages once the bridge has been opened to traffic.

Nevertheless, in this case some undesirable effects during launching would have appeared if the lower bracing was designed and installed during these construction stages. For this reason, the design did not include the lower bracing in this bridge. From the analysis carried out in service limit state, the lower bracing has benefits for the design of curved steel I-girder bridges. With these structural and geometrical configurations, and provided that either the construction method is with cranes, or the bracing is added in an existing bridge which needs structural improvements, the addition of lower bracings can improve the structural behavior of the bridge and cut the construction- and/or investment costs.

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