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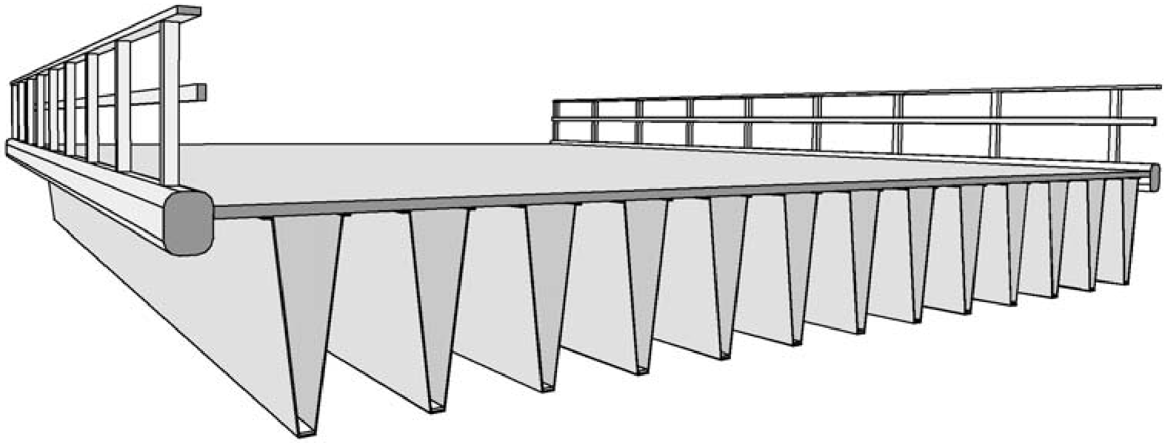

Industrial Bridge Engineering

– Structural developments for more efficient bridge construction

PETER HARRYSON

Department of Civil and Environmental Engineering
Structural Engineering – Concrete Structures
CHALMERS UNIVERSITY OF TECHNOLOGY
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THESIS FOR THE DEGREE OF DOCTOR OF PHILOSOPHY

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Cover:
Outline of the *i-bridge* concept, a perspective sketch illustrating the major components of the bridge.

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ABSTRACT

In recent years, growing concern about the deficiencies and lack of efficiency in the construction industry has highlighted the need of research in order to make substantial improvements, rectify incongruities and succeed in progressive development. For bridges, the advantages that can be expected from an industrial construction process are especially interesting. The multidisciplinary research presented in this thesis investigates different means of creating an industrial process for bridge construction while emphasising the vast importance of structural engineering in combination with industrial issues, i.e. industrial bridge engineering. Applications of new or approved techniques, materials technology and developments, methods of design and analysis, as well as construction methods are important areas that have been considered. The increasing utilisation of information and communication technology (ICT), along with more advanced computer-based analysis and simulation methods, are contemporary trends. Seemingly, an important key to the successful construction concepts of tomorrow is to combine these factors into an efficient industrial process.

One objective in the presented work has been to establish the foundations for attaining such a new process – basically through analysis of the current bridge construction process – as well as to determine the required improvements needed to provide a framework for a new industrial process. Deficiencies in the traditional bridge construction process have been recognised, as have the underlying driving forces of change. Three cornerstones of industrial bridge construction have been identified – *process development, product development and productivity development* – and technical necessities have been investigated.

Furthermore, a study of an innovative jointing technology for connections between prefabricated concrete elements in bridges has been conducted. The aim has been to design a joint that makes the surrounding elements continuous, but still a very small joint that is easy and fast to perform, and thus highly suitable for use in industrial bridge concepts.

In addition, a feasibility study of a novel industrial bridge concept has been undertaken. The concept embraces ultra-high-performance steel-fibre-reinforced concrete in composite action with fibre-reinforced polymers. Several laboratory tests and finite element analyses have been conducted. The main focus has been to investigate new or approved techniques from a design viewpoint as a continuation of recent materials developments, while also considering industrial aspects and construction characteristics such as production methods, assembly, etc.

Keywords: Bridges, industrial construction process, product and productivity developments, design, structural developments, materials developments, structural joints, steel-fibre-reinforced ultra-high-performance concrete, fibre-reinforced polymers.

Industriell brobyggnadsteknik
– konstruktionstekniska tillämpningar för ett mer effektivt brobyggande
PETER HARRYSON
Institutionen för Bygg och Miljö
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SAMMANFATTNING

På senare tid har en växande medvetenhet om felaktigheter och brist på effektivitet inom byggbranschen gjort att forskning för att förbättra situationen och rätta till brister blivit högaktuell för att branschen ska kunna gå vidare i en framåtskridande utveckling. För broar är de fördelar man kan förvänta sig av en industriell byggprocess särskilt intressanta. I den multidisciplinära forskning som presenteras i denna avhandling utreds olika medel för att skapa en industriell byggprocess för broar med tyngdpunkt på den stora vikten av konstruktionsteknik i kombination med industriella ämnesområden, dvs. industriell brobyggnadsteknik. Tillämpning av ny eller beprövad teknik, materialutveckling, konstruktions- och analysmetoder likväl som byggmetoder är viktiga områden som har beaktats. En tilltagande användning av informations- och kommunikationsteknologi (ICT) i kombination med mer avancerade datorbaserade analys- och simuleringsmetoder är en tydlig pågående trend. En viktig nyckelfråga för morgondagens framgångsrika byggkoncept kan vara att kombinera dessa faktorer i en effektiv industriell byggprocess.

En av målsättningarna med det presenterade arbetet har varit att etablera en grund för fortsatt arbete med att utveckla en ny industriell byggprocess – främst genom analyser av den traditionella brobyggnadsprocessen – men också att utvärdera de erforderliga förbättringar som krävs för att utveckla ett ramverk för industriellt brobyggande. Bristerna i den traditionella brobyggnadsprocessen har uppmärksammats liksom de underliggande krafter som verkar mot en förändring. Tre hörnstenar för ett industriellt brobyggande har identifierats – *processutveckling, produktivitetsutveckling och produktutveckling* – och nödvändig teknisk utveckling har utforskats.

Utöver detta har en studie av en innovativ sammanfogningsteknik för prefabricerade betongelement genomförts. Målsättningen har varit att utveckla en fog som är liten, lätt och snabb att utföra och som samtidigt gör de sammanfogade elementen monolitiska. Sådan fogteknik skulle innebära en förbättring av effektiviteten på arbetsplatsen och alltså passa väl in i industriella brokoncept.

Dessutom har en förstudie av ett nytt industriellt brokoncept utförts. I brokonceptet ingår ultrahögpresterande stålfiberarmerad betong i samverkan med fiberarmerade polymerkompositer. Flera laboratorieförsök och finita elementanalyser har genomförts. Förstudiens huvudsakliga fokus har varit att utforska ny eller beprövad teknik ur konstruktionshänseende som en förlängning av senare tids materialutveckling, med beaktande av industriella aspekter och produktionsförutsättningar för brokonceptet.

Nyckelord: Broar, industriell byggprocess, produkt- och produktivitetsutveckling, konstruktionsteknik, konstruktionsteknisk utveckling, materialutveckling, detaljutformning, betongfog, ultrahögpresterande stålfiberarmerad betong, fiberarmerade polymerkompositer.

LIST OF PUBLICATIONS

This thesis is based on the work contained in the following appended papers, referred to by Roman numerals in the text.

Paper I — Harryson, P.: Explorations of Different Means to Achieve an Industrial Process for Building Bridges; part I – implications out of the current process, *Nordic Concrete Research,* Publication No.30, 2/2003, pp 31-52.

Paper II — Harryson, P.: Explorations of Different Means to Achieve an Industrial Process for Building Bridges; part II – evaluation of contemporary strategies, *Nordic Concrete Research,* Publication No.30, 2/2003, pp 53-68.

Paper III — Harryson, P. & Gylltoft, K.: High Performance Joints Between Prefabricated Traffic Slab Elements for Industrial Bridge Construction, *High Perfomance Materials in Bridges* (proceedings from the International Conference on High Performance Materials in Bridges and Buildings, Kona, USA, 2001), edited by Atorod Azizinamimi, Aaron Yakel and Magdy Abdelrahman, ASCE, 2003, pp 253-264.

Paper IV — Harryson, P.: High Performance Joints for Concrete Bridge Applications, *Structural Engineering International,* Volume 13, 1/2003, pp 69-75.

Paper V — Harryson, P.: The *i-bridge*, a novel bridge concept, Feasibility studies embracing industrial bridge engineering. Submitted to *Structural Engineering International*

Paper VI — Harryson, P.: Bond between Fibre Reinforced Concrete and Fibre Reinforced Polymers, Experimental Study. Submitted to *Materials and Structures*.

Paper VII — Harryson, P.: Laboratory test and finite element analyses of a prototype bridge beam. Submitted to *Nordic Concrete Research*

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Harryson, P.: *Utmattningprovning av fog i högpresterande fiberbetong för prefabricerade brobaneplattor* (in Swedish, "Fatigue testing of joint in high performance fiber reinforced concrete for prefabricated bridge deck slabs"). Report 00:2, Department of Structural Engineering, Chalmers University of Technology, Göteborg, 2000, 56 pp.

Harryson, P.: *Böjprovning av fog i högpresterande fiberbetong för prefabricerade brobaneplattor, statisk belastning* (in Swedish, "Bending testing of joint in high performance fiber reinforced concrete for prefabricated bridge deck slabs, static loading"). Report 99:1, Department of Structural Engineering, Chalmers University of Technology, Göteborg, 1999, 76 pp.

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CONTENTS

APPENDED PAPERS I – VII

Paper I Harryson, P.: Explorations of Different Means to Achieve an Industrial Process for Building Bridges; part I – implications out of the current process, *Nordic Concrete Research,* Publication No.30, 2/2003, pp 31-52.

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Paper VII Harryson, P.: Laboratory test and finite element analyses of a prototype bridge beam. Submitted to *Nordic Concrete Research*

Preface

This work that has been carried out at Chalmers University of Technology and that eventually resulted in this thesis has been conducted on part time basis over a period of years, the last part starting in late 2002. The work has been rather scattered from time to time when focus has been on other assignments, while other periods have been extremely intensive. The research project has been a part of the research consortium ROAD, BRIDGE & TUNNEL's Infrastructure program and it has been financed by the Swedish Road Administration (SRA), VINNOVA (the Swedish Agency for Innovation Systems) and Chalmers University of Technology.

I am grateful to my supervisor at Chalmers, Professor Kent Gylltoft, for his valuable advices, support and encouragement. In addition, the 'project board' consisting of Kent Gylltoft, Anders Huvstig (SRA) and Hans Bohman (SRA) has contributed with valuable discussions. I also wish to express my gratitude to all my colleagues both at Chalmers and at the Swedish Road Administration, especially Ingvar Andersson and Björn Ållenberg for supporting me during this time. In addition, I am grateful to research engineer Lars Wahlström and the inventive Nils Nilsson for helping me overcome all obstacles and finally realizing all laboratory tests. Thanks are also due to my namesake, the Swedish actor Peter Harryson – whose (unwanted?) phone calls often are misdirected to me – for making my name so easy for everyone to remember.

And finally, to my family, who recently have had to put up with an absent-minded father and husband. Thank you for your ever-lasting patience with me. To my children I wish to declare that they will have a substantially better access to their father, starting from now. To Moa, I will respond instantaneously and you can stop being angry with me. Among other things, we have many amusing hours of car driving lessons to look forward to. To Hampus, we have a lot of football games to play and I do believe you have promised to introduce me into the World of Warcraft. To my little one, Olle, I can only say stop growing up so fast, because we have a lot of fun things to do! Finally to my beloved wife Maria, who embraces an ocean of empathy. You are the hub of my life!

Göteborg, April 2008

Peter Harryson

Notations

Latin upper case letters

A	Area
D	Stiffness of interfaces
E	Modulus of elasticity
G	Modulus of shear
N_b	Amount of fibres bridging the failure surface
V	Volume fraction

Latin lower case letters

f	Strength
g_f	Fracture energy
n_b	The fibre efficiency factor

Greek lower case letters

δ	Displacement
γ	Partial safety factor
ν	Poisson's ratio
σ	Normal stress
τ	Shear stress

Subscripts

11	Normal direction
22	Shear direction
c	Concrete or Compression
cr	Cracking
f	fibres
k	Characteristic value
m	Material or Mean value
n	Normal direction
s	Steel
t	Tension
u	Ultimate value
y	Yield value
x	Co-ordinate
y	Co-ordinate
z	Co-ordinate

Abbrevations

CIC	Computer-integrated construction
CFRP	Carbon-fibre-reinforced polymers
CRC	Compact Reinforced Composite
FE	Finite element
FEM	Finite element method
FOS	Fibre optic sensors
FRP	Fibre-reinforced polymers
ICT	Information and communication technology
GFRP	Glass-fibre-reinforced polymers
IM	Intermediate-modulus (for carbon fibres)
JIT	Just-In-Time
PAN	Polyacrylonitrile
PPP	Public-private-partnership
SLS	Serviceability limit state
TFV	Transformation, Flow, Value; (theory of production)
UHPSFRC	Ultra-high-performance steel-fibre-reinforced concrete
ULS	Ultimate limit state

1. Introduction

1.1 Background

This thesis is the result of work that has been done as a response to the increasing demand for more efficient and competitive ways of constructing bridges. The development in bridge construction has not been very progressive in Sweden over the last decades; new techniques and methods have seldom been presented. Traditionally, bridges are usually cast *in situ*, involving a massive use of manpower and techniques that may be characterised as more or less craftsman-like. Prefabricated concrete elements, for example, are not used very often. This is not the case in the international field, where many industrial concepts of bridge construction have evolved, as summarised in Paper I. However, many of these concepts have a bearing only on larger bridges. Apart from what is presented here, further elaboration of the background for the research project can be found in Harryson, Gylltoft & Norén (1999), Harryson & Gylltoft (2001), and in Paper I and Paper II.

Although the research field is novel in this form, the idea of industrial construction is not new. Already in ancient Greece instances occurred where famous structures were erected with prefabricated components of stone, as reported by Warszawski (1999). Realisation of the industrial ideas in construction has attracted researchers and practitioners since the heyday of the Industrial Revolution. In fact, already in the 19th century attempts with prefabricated housing were made in Sweden, and in the 1930s there was actually a house factory with a moving belt in the USA – although it never became a success. The same can be said of many other attempts made in the area of industrial construction.

Adding to the inefficiency is a significant amount of deficiencies and peculiarities in the current construction process. The Swedish commission of inquiry on deficiencies in the construction sector, reported in SOU 2002:115 (2002), represents a culmination of the discussion of these problems in Sweden. Following the commission's report many developments have been initiated mainly in the residential building sector, while no large progress can be reported from the civil engineering sector in aspects of industrial developments.

It is easy to understand why an industrial process for bridge construction is attractive. In view of recent advancements made in the fields of materials science and technology, design and analysis methods, production techniques, etc., as well as the rapid development in information and communication technology (ICT), there is a vast potential of rationalising construction represented by a contemporary industrial process. It is somewhat more difficult to imagine what the resulting process would be and the features of the products that would be manufactured, but some likely characteristics can be foreseen. However, the question of how and in what way it is going to be realised is more diffuse. Thus, a major objective in the first part of this project has been to set forth the boundaries for a framework of industrial bridge construction and to evaluate directions with the highest potential for improvement. In addition, concepts of detailing have been identified as key factors to promote efficient construction; therefore the second part of the study is committed to this issue. Moreover, since the lack of industrial bridge concepts prevents the discussion from being directed towards industrial issues, efforts have been devoted to identifying a far-reaching industrial concept utilising the essence of contemporary developments. Hence, the power of a good example can be used as a lever arm to encourage developments in construction. Some of the interacting components connected with Industrial Bridge Construction are shown in Figure 1. Summing up, since there are so many ways of improving the business of bridge construction, this project certainly has been a journey of exploring different means heading towards an industrial process for building bridges.

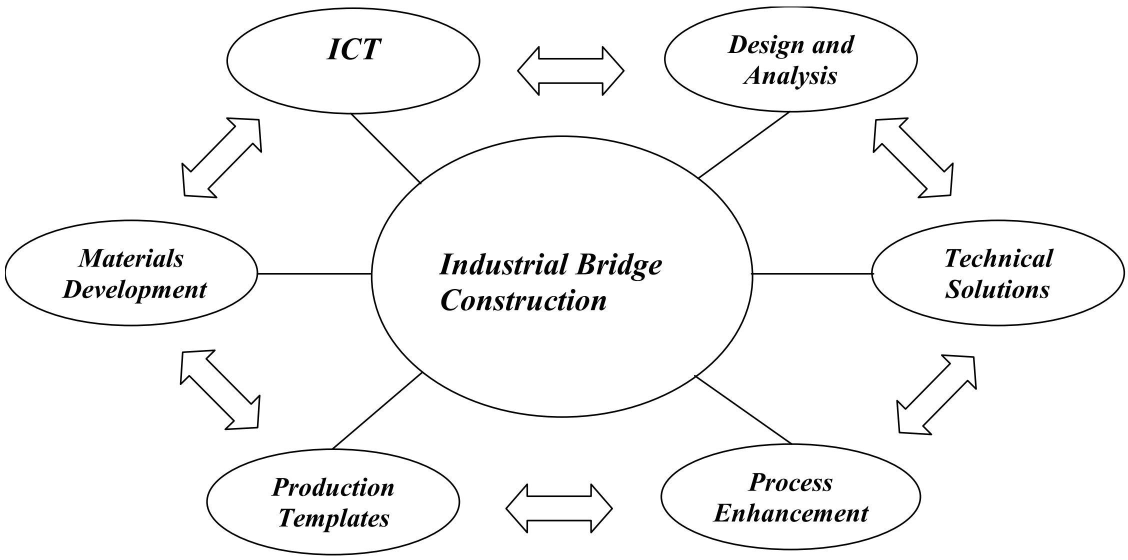


Figure 1. Some of the interacting components connected with Industrial Bridge Construction.

1.2 Aim and scope

The main questions to be answered by the work presented in this thesis are:

(1) Can an industrial construction process be one possible solution to the deficiencies faced in bridge construction?
(2) What are the most effective ways to create more efficient bridge construction and to enhance developments?
(3) How can structural engineering and industrial bridge engineering contribute to realising new industrial bridge concepts?

The main hypothesis is that an industrial process of bridge construction can be a feasible solution, but only if a comprehensive overall approach is adopted where all parts of the process are included and integrated into the industrial bridge concept. The aim has been to answer the main questions by investigating and exemplifying possible means of more efficient bridge construction. Hence, the expected result is that the same funding will be adequate for further infrastructures if the objectives are achieved; in other words, there will be more bridges for the same money. Research areas involved are, for example, materials science, structural engineering and design, production methods and techniques, process development, and information and communication technologies (ICT). The research in these disciplines, however, is not usually connected with regard to all parts of the building process. It is of course impossible to investigate all interrelated research areas in depth. Thus, the aim has been to penetrate all of them in order to evaluate their potential benefits when applied in an industrial process for bridge construction, while focusing on structural engineering and design in the deepened studies.

The aim of the work can naturally be broken down to address different levels according to the different parts of the work. In an overall perspective, the ambition of the research project is to enhance developments in design and construction of bridges, with a more direct view towards industrial construction of bridges.

The aim of the conducted process analyses is to lay the foundation for the suggested improvements needed to provide a framework for the industrial processes.

The aim of the study of an efficient jointing technology for concrete is to demonstrate how new developments can be utilised in bridge construction.

The aim of the feasibility study of a novel bridge concept, the *i-bridge,* is to illuminate the possibilities given by new technology, new materials and other advancements when developing concepts of industrial bridge construction. Hence, the intention is to express how structural engineering can contribute to enhancing the effectiveness of the overall construction process and encourage development.

1.3 Research approach

Because of the multidisciplinary features of the work, an important issue has been to find suitable research approaches in order to penetrate the different fields involved. Hence, the research methods differ to some extent between the different studies.

In the process analyses, the main research method chosen was an explorative study to obtain basic knowledge of root problems and suggestions for improvements.

The study of a novel jointing technology for concrete comprises the combination of laboratory tests and finite element (FE) analyses as the major research approach.

In the feasibility study of the *i-bridge* concept, the main focus has been on conceptual design and industrial characteristics while utilising the materials developments in the field of fibre-reinforced polymers (FRP) and ultra-high-performance steel-fibre-reinforced concrete (UHPSFRC). This has been combined with FE analyses on different levels of accuracy and selected laboratory tests to examine the structural behaviour and the material properties. Additionally, in the assessment of the prototype beam, the laboratory load test was investigated with FE analyses.

1.4 Limitations

No special limitations were stated beforehand regarding technical developments, materials, application of new developments, etc. Hence, a comprehensive approach and freedom of choice in these matters were considered a prerequisite for a successful outcome.

An important limitation, though, has been to focus primarily on structural engineering and design issues and not to go into detail in the corresponding research areas. Furthermore, the studies refer primarily to the situation in Sweden, although most of the work is applicable also in an international perspective. Moreover, the project should focus on short- and medium-span road bridges. Additionally, the feasibility study of the *i-bridge* concept is by its nature limited to the initial conceptual stages. Thus, further investigations are needed in subsequent phases.

1.5 Outline

The thesis is based on the work contained in the appended seven Papers I–VII. The purpose of the presentation in the thesis is to put the conducted work into perspective and to connect the parts with logical links between the different research topics. While the actual studies are presented in the attached papers, a comprehensive summary is offered here as well as further elaboration of some of the discussions.

Besides the introductory part, which is presented here in Chapter 1, and the concluding parts, which are presented in Chapter 5, the content can roughly be divided into three branches, which is the disposition followed in the thesis.

The first branch, comprising Paper I and Paper II, is presented in Chapter 2. This initial study deals with the overall research problem, the necessary infrastructure of an industrial process, and how the different components are linked. Paper I is devoted to comments and analysis of the current bridge construction process in comparison with developments in the direction of a new industrial process. A general introduction to the research topic can also be found. In Paper II, the emphasis is turned towards problems and opportunities in an industrial context, where the underlying driving forces acting in favour of an industrial process are investigated and the necessary prerequisites are evaluated. In addition, some priorities and conclusions about the different potential developments are drawn. In these papers the foundation for the work is laid, and feedback from the literature survey is described, as well as an overall investigation of different technical aspects. The emphasis is on erecting guidance for a new industrial bridge construction process by means of analysing the current construction process. Thus, in a very broad approach, different means to achieve such a process are explored.

The second branch, comprising Paper III and Paper IV, concerns the important issue of detailing and is presented in Chapter 3. The articles describe the development study of an innovative jointing technology for structural connections between prefabricated bridge deck slab elements. The area of detailing in general, and jointing in particular, has been recognised as one of the most important issues that have a major impact on the competitiveness of a specific bridge concept – especially when prefabricated concrete elements are included. Although, by necessity, some of the contents are overlapping between these papers, the focus in Paper III is a presentation of the laboratory tests performed as well as the results from the tests, while the emphasis in Paper IV is on evaluating the FE analysis conducted.

In the third branch, consisting of Papers V–VII, a feasibility study of a novel bridge concept, the *i-bridge,* can be found. This part is presented in Chapter 4. The concept embraces ultra-high-performance steel-fibre-reinforced concrete in composite action with fibre-reinforced polymers. Several laboratory tests and FE analyses have been conducted.

In Paper V, a description of the concept and the conceptual design and FE analyses are presented.

Paper VI presents selected laboratory tests in the form of an experimental study of the bond between concrete and glass-fibre-reinforced polymers, to apprehend the behaviour and material properties.

In Paper VII, the assessment of a prototype beam with laboratory load test and evaluation of FE analyses can be found.

Since the presentation is related to the appended papers, references are kept very sparse, apart from general reference to the papers. Direct references are made only if explicitly needed. However, the reference list in Chapter 6 includes all literature, including that from the papers.

1.6 Original features

The *i-bridge* concept is an original concept that has emerged from the studies of developments for potential use in industrial bridge concepts. The same applies to the prototype beam that was load-tested in the laboratory. In addition, some of the means used for enhancing the bond between concrete and fibre-reinforced polymers in the experimental study have not been seen before. The performance and the numerical investigation of the fatigue tests in the joint study are also original. Moreover, the general research approach combining process analyses and structural engineering with corresponding research areas is, to the author’s knowledge, seldom used.

2. Contemporary process and industrial implications

2.1 The traditional bridge construction process and industrial changes

2.1.1 Deficiencies in the current construction process

The shortage of efficiency development in the construction industry compared to other industries has been highlighted in recent years along with other deficiencies in the current construction process. Hence, it is these problems that form the basis for this thesis.

Two of the most apparent differences in the construction industry compared to other industries are the long service life of the products and the fact that the eventual production location is fixed whereas the production facilities (the 'factories') traditionally have been mobile. Moreover, the high safety requirements and the extensive involvement of society are important factors generally characterising the construction industry.

A comprehensive analysis of the current bridge construction process has been performed and is presented in Paper I. A survey of some important problems encountered in the analysis is summarised in Paper II and presented in Table 1.

Table 1. Problems to be dealt with in the current bridge construction process.

Problems in the current bridge construction process:

- Large involvement of society.
- Large fluctuations in construction volumes.
- Great variations in the outline of bridges.
- Small series and small volumes per bridge type.
- Large amount of participants in the process.
- The 'peculiarities' of construction.
- Lack of a consistent production theory.
- Diffuse customer perspective.
- Relatively low technical level.
- Relatively low entrance resistance.

The deficiencies established from the process analysis were to a large extent the same as subsequently was concluded in SOU 2002:115 (2002), the Swedish commission of inquiry on deficiencies in the construction sector. This commission constituted a climax of the discussion on these problems and, since the report, many developments have taken place in the housing sector (see e.g. Lessing (2006)) while no large progress regarding industrial developments is to be noted in the civil sector.

Although the circumstances indicated in this chapter represent great obstacles, they should not serve as an excuse for preserving the current situation. On the contrary, they ought to enhance and stimulate efforts to overcome the problems.

2.1.2 Drivers of change

As implied in the previous chapter, the need of improvements originates from the deficiencies encountered in current bridge construction. There are, however, also important underlying driving forces of change influencing the construction industry. These can be identified on a global level, on a regional or national level, as well as on a company level, and some of the most important factors are briefly summarised in the following.

Globalisation in general is among the most important factors influencing the conditions for construction on a global level. As politics, social development, communications, economics, demands and competition etc. become ever more global, the construction industry is inevitably affected by these prerequisites. Furthermore, demographics are a major issue likely to influence construction. As populations continue to grow substantially in developing countries, while declining in developed countries, new solutions in construction are liable to emerge. Another demographic factor is the ageing of populations in developed countries, which will create new patterns of living and, not least, an unwillingness to undertake construction work with harsh and unsafe conditions. A third important factor on the global scale is the environmental situation and especially the global warming problem, which certainly will impose restraints also on the construction sector.

The issue of labour health and safety is currently a strong driver of change in many developed countries. Another important aspect in many regions is the need to attract bright young people into a career in the construction industry, for example in Japan, where one factor driving developments is the labour shortage (see Li (1995)).

On the company level, the ever-increasing need of improving profitability in order to become attractive for investors on the market will intensify the search for more efficient construction methods. Additionally, a growing concern of stakeholders other than stockholders, such as labour, society, customers, etc., also seems likely to play an equally important role. Thirdly, to survive the competition, the companies strive to reach world class. Although the construction business traditionally has been a local market, the erased borders between countries and regions mean that this trend can be expected to emerge in the construction industry as well.

2.1.3 Activators of changes

The literature mentions two basic, principal ways to bring about such changes, indicating who is to be the driving part in terms of developments and implementation of the new processes. These are the *technology-push* approach, whereby new technical solutions superior to the old ones will have to be presented in order to create a demand, and the *market-pull* approach which means that the demands from the market will result in new solutions. In the case of bridge construction, however, it is not possible to distinguish sharply between the two approaches, due to the absence of a clear-cut customer perspective and to the intense involvement of society and public commissioners.

On the one hand, it is easy to perceive that it must be the industry providing the products, i.e. the contractors or even the consultants, which will have to take the lead by developing new products, since there is much to gain from competitive advantages.

However, due to the large uncertainties and the heavy investments connected with such developments, this is not likely to take place without a clear incentive.

On the other hand, most of the specific demands made in the process originate from the public commissioners, with a significant bearing on their administrative issues. Their role as both commissioners and administrators puts them in a special situation; they are obviously in a position to influence the features of the process, but this is usually not done.

Hence, a combination of the two approaches seems to be a feasible solution to question of who is to lead and implement the changes. The incentives and much of the framework for the overall process ought to be presented by the public commissioners, while the specific ideas and new products are best developed by the industry.

2.1.4 Industrial implications

There are different ways to implement industrial processes for bridge construction. Base lines for a framework of industrial bridge construction have been given in Paper I and Paper II. Some fundamental prerequisites have to be fulfilled in order to reach sustainable industrial development in bridge construction. There must be propositions about the organisation of a new industrial construction process, since the traditional process is not particularly suited for industrial purposes, and a consistent theory of production will have to support the industrial ideas. Additionally, ideas of industrial concepts, utilising appropriate developments in materials, design and production methods, must be presented.

Moreover, a clear-cut customer perspective is a necessity in a new industrial process, where the commissioner is a client representing the real customers and society, instead of a pure commissioner perspective. A critical item in the process needing special attention is procurement, since it is the single most important factor for motivating the contracted participants. Furthermore, a comprehensive interconnection of the parts by means of information and communication technology (ICT) is needed in order to organise and extract all advantages from the process, compare e.g. Olofsson et. al (2004). Hence, the degree of integration of ITC into the systems will significantly influence the actual outcome from all parts of the process.

Finally, the implementation of industrial concepts must be the result of strategic decisions and consistent work in accordance with specified plans. It is essential that the whole organisation is motivated and committed to take part in the work. Thus, the industrial ideas must be present in each part of the organisation as well as throughout every project concerned, from initial engagement to complete realisation. All these factors have been discussed in Paper I and Paper II.

In order to avoid the problems faced in Swedish industrial housing construction, a more stepwise evolution for the bridge sector seems recommendable. The development and investments in industrial housing concepts were boosted in Sweden after the massive criticism in SOU 2002:115 (2002), but just recently after only a few years of production several of the most far-reaching building systems were terminated – for economic reasons, but also seemingly due to lack of endurance. Hence, a combination of market-pull and technology-push is essential as argued above, where the demand for development and new solutions as well as the framework for the overall process is put in place as a first stage. The implementation phase currently seems to be the most critical part of the evolution process, since new ideas have large problems in reaching the market.

Thus, research must focus more on demonstrating the performance, increased efficiency, and potential cost savings of the developments. Nevertheless, in view of the conservatism that presently characterises the construction business, regardless of how large the potential is, it remains possible that new concepts must prove profitable from Day One in order to be able to break the surface into the market. The branch of bridge construction that currently seems most adapted to the industrial ideas is the wooden bridge sector, where many bridge types are produced in an industrial manner.

2.2 Characteristics of industrial bridge construction processes

2.2.1 Industrial features

Uninformed discussions often compare industrial construction with large, tedious, straight-in-line prefabricated concrete buildings, as was the former perception from the era of mass housing production. This resemblance is no longer valid for contemporary industrial concepts, since more recent approaches are able to differentiate the needs of customers. Some of the characteristics and methods mentioned in connection with industrial construction in the literature are standardisation, modularisation, prefabrication or off-site fabrication, as well as on-site fabrication, pre-assembly, mechanisation, automation, and the use of different building systems. Most of these features also apply to bridge construction. The actual features will of course differ significantly between different specific concepts of industrial bridge construction, for example if an industrial on-site approach to production is adopted or if the concept is extensively based on prefabricated components produced off-site and assembled on site.

The goal of every industrial process is to generate products at a lower cost, or to fabricate products of higher quality at the same cost. An optimised process might even realise both objectives simultaneously, i.e. higher quality at a lower cost. Moreover, to reduce the overall construction time without putting the quality at risk is another important objective. In general, simplicity and repetition often make a reliable recipe for successful concepts.

Although not totally consistent, there are two different approaches to industrial developments: one mainly refers to modernising of traditional construction, while the other emphasises new development from scratch without unnecessary connections to the elderly construction process. Hence, in this thesis the term ‘industrial’ is defined as industrial development referring to contemporary solutions, and the term ‘industrialised’ is defined as development related to transforming traditionally craft-based methods into something more modern.

Furthermore, the term ‘industrial engineering’ is defined as an approach emphasising the vast importance of structural engineering in combination with industrial issues. That means adding the industrial requirements to the existing requirements of structural engineering, which are usually considered to be structural stability, economy, durability, environmental aspects, sustainable development, aesthetics, and social responsibility, in the design of structures.

A high level of technology is necessary since it is an evident trend that production, other than service and technically demanding manufacturing, is located in low-wage developing countries. While the peculiarities of construction make it difficult to e.g. mass-produce large components in faraway locations, the increased mobility of the workforce can result in similar effects for the construction industry, as has been seen in recent years in Sweden.

Even though the ideas offered are viable primarily for bridges, they could very well be extended into other areas of construction. For example, the work includes the six chief aspects to be regarded in developing industrial systems, as summarised by Warszawski (1999): physical performance (stability, strength, maintainability, etc,), architectural design (aesthetics, etc.), technology (selection of materials, production methods, jointing, finishing techniques, etc.), management (planning and coordination of production, quality control, etc.), economics (forecasting of demands, selection of the most profitable method and optimal location of production, etc.) and marketing (sales engineering, effective contracting, etc).

2.2.2 Advantages and drawbacks

The main advantage that can be expected from industrial construction of bridges is an increased customer value resulting from a substantial reduction of waste, by practically eliminating the uncertainties and the peculiarities of construction. Other potential advantages are more efficient and rational construction resulting in shorter time of construction, improved employee performance due to better working conditions, and better use of resources from a public economic point of view. Moreover, a comprehensive process will provide better control facilities, which might yield possibilities to predict and reduce the environmental effects of construction, to take sustainability aspects into account, and to foresee and reduce the need for maintenance.

The conducted process analysis, merged with implications of industrial process improvements in bridge construction, has resulted in the performance specification presented in Paper I and in Table 2.

The major disadvantage of industrial construction, as noted e.g. by Koskela (2000) and by Warszawski (1999), mainly referring to prefabricated concrete elements, is the increased complexity of the process. However, processes unable to deal with complex situations are obviously far from optimised, e.g. industrialisation imposed on a traditional process. It is important, though, to address this deficiency and to organise the process so that the advantages strongly outweigh the drawbacks.

Furthermore, the large investments generally required, and the difficulty – commonly persisting in construction – of protecting immaterial property rights, are other problems to be aware of. The importance of protecting immaterial rights such as patents is evident, since it provides opportunities for niche products to be exposed on a larger market, justifying investments in research and product development as well as in production facilities. This could also be a solution to the problem of insufficient production volumes to motivate large investments.

Table 2. Performance specification for industrial bridge concepts.

Basic demands: Demands that can be stated for any construction project. - Functionality, safety, serviceability and comfort criteria. - Economic and efficient construction, and delivery within time schedule. - Quality and durability aspects, lifetime assessment, maintenance reduction and environmental concern.
Advantages creating added value: Advantages (or bonus) on behalf of an industrial process. - Reducing overall construction time and cost. - Flexible process, both during construction and afterwards, with no disturbance due to construction work. - Optimisation of design and enjoyable experience of the aesthetics. - “Extras” creating goodwill, e.g. shorter delivery time, easier procurement and less administration. - Improved quality aiming at zero mistakes and consequently no guarantee work. - Continuous development and improvement of process, products and productivity.
Complementary demands for an industrial process: Besides the first general demands, the other examples depend to a great extent on the features of each specific concept; thus there is no general applicability. - Reduction or elimination of wastes, uncertainties and peculiarities. - Simple and rational construction or manufacturing, resulting in lowered construction cost. - Easy, fast and straightforward design, integrated in the process and simplicity in aesthetic adjustments. - Improved working conditions for employees. - Simple and minimised work on site and all work carried out in sequence, thus avoiding later rework. - Simple and light elements to minimise transportation, heavy lifts etc. and to provide efficient assembly. - Open systems with standardised interfaces of components. - For prefabricated systems, a high degree of prefabrication is aimed at, preferably with no on-site work except erection and assembly.

An important obstacle concerns the problems due to the fact that bridges usually are parts of road or railway connections. Bridge projects, especially small and medium-span bridges, are frequently included in larger road contracts where they play a minor but not insignificant role. As a consequence, co-ordination between road and bridge construction activities is very important in many cases. Problems may also arise as a result of the procurement being governed by road requirements or different procurement forms for roads and bridges. The solution seems to be to apply the industrial process to the overall project, whereby the same industrial ideas are shared for both road and bridge construction.

2.2.3 Production theories

From the process analysis and the literature, an absence of a consistent production philosophy can be noticed in many cases. If a philosophy is used at all, it tends to be the traditional transformation theory (i.e. transformation of input to output), while more recent ideas are seldom implemented. Hence, it is a vital issue that new industrial bridge concepts apply contemporary production theories that are both practically and scientifically founded. There are many relevant production philosophies that can be fetched from other industry sectors, as is often concluded in benchmarking studies.

One comprehensive production theory for the construction industry is the TFV concept, presented by Koskela (2000). It has incorporated many of the contemporary production views from other industries, and it could be utilised for industrial construction processes. Hence, a concentrated summary of the theory is presented here.

The general scheme of the TFV concept is the integration of the three prevalent views on manufacturing into a comprehensive theory of production. The three views are:

- *Transformation* (T), i.e. the traditional view of manufacturing as transformation of input into output. Here the emphasis is to realise the transformation as efficiently as possible by decomposing the production into tasks and minimising the cost of each task.

- *Flow* (F), which means that the production is regarded as a flow through the process. The main principle is to reduce the share of non-value-adding activities (waste), leading to compressed lead time, reduced variability in the products, simplicity, increased transparency and increased flexibility. Just-In-Time (JIT) and lean production are methods of the F type.

- *Value generation* (V), whose main principle is to improve customer value by ensuring that all requirements are captured, ensuring the flow-down of customer requirements, ensuring the capability of the production system, and measuring value. This is the origin of the quality movement and other customer-oriented methods.

In short, the production is looked upon as a flow connecting the transformation parts, while focusing on creating customer value through the whole process, as visualised in Figure 2. The theory is also used as a tool in the analysis of the current bridge construction process where it reveals some interesting aspects, as presented in Paper I.

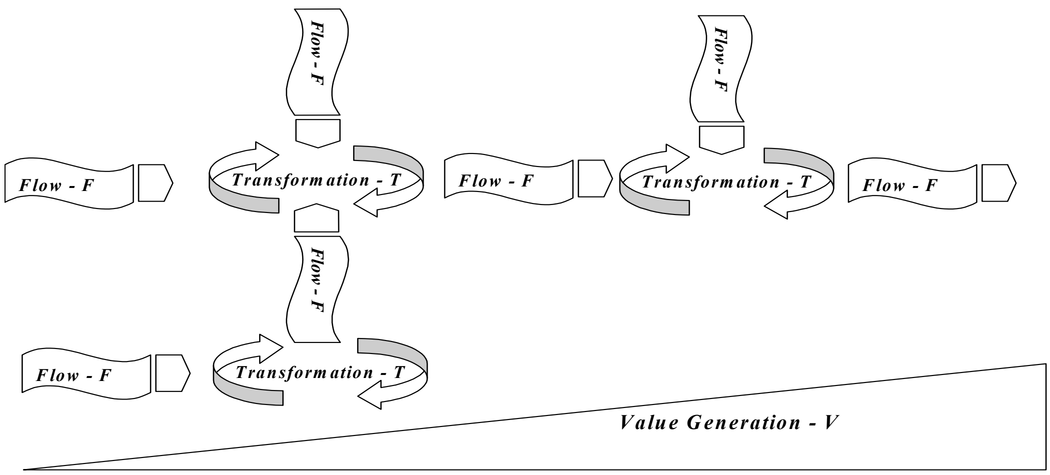


Figure 2. Generalised illustration of the TFV scheme.

2.2.4 Cornerstones of Industrial Bridge Construction

To realise industrial concepts of bridge construction, an intensive correlation and co-operation between different disciplines is required, as has been pointed out in e.g. Paper I and Paper II. Subsequently, there will be aspects bearing on the organisational or managerial domain as well as aspects stressing the technical domain, but there will be few aspects which solely emphasise just one of the domains. The cornerstones of industrial concepts of bridge construction have been identified as the three P's – *process development, productivity development and product development* – forming a progressive, continuous circle of developments integrated by information and communication technology (ICT), as visualised in Figure 3.

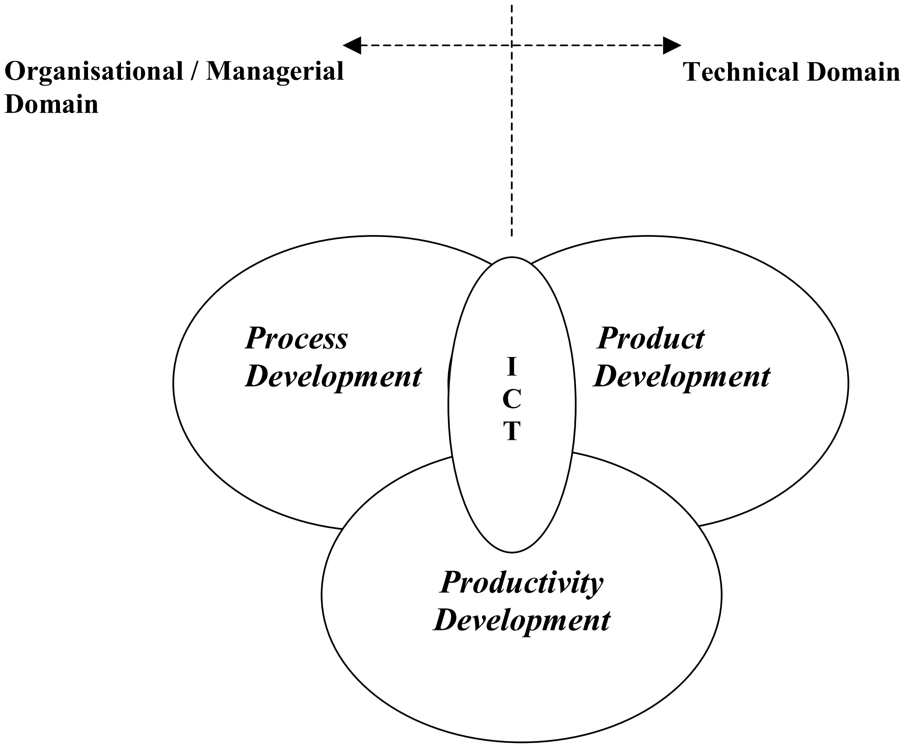


Figure 3. The cornerstones of Industrial Bridge Construction – the three P's.

Appropriate weight must be put on each of the cornerstones in order to extract all benefits from industrial concepts. For example, new products with excellent performance have small chances of becoming successful unless they are produced efficiently and channelled by an efficient process giving access to the market. Likewise, an exceptional production technique has no relevance for a product that is undeveloped and not demanded on the market, or if the process is unable to provide a connection to the market. Hence, it can be concluded that the cornerstones have their 'centres of gravity' in different domains to some extent. While *process development* seems heavily weighted in the organisational/managerial domain, the emphasis of *product development* lies in the technical domain, and *productivity development* tends to stress both domains about equally.

Concerning *process development,* a new industrial process is likely to initially contain the basic construction aspects – the production and the design stages. While the two other cornerstones, productivity and product development, are embraced within the process, the process will have to take all activities and all participants into account as well as constituting the infrastructure to support and govern the actual production. The main emphasis of the *process development* is flow management (the F view in the TFV philosophy), aiming at eliminating waste and non-value-adding activities in all stages. The implementation of an industrial process will provide the first part of the immediate competitiveness of the bridge concept, while the continuous process development will enhance the performance and govern the competitiveness in the long run. When the industry and the clients become more acquainted with industrial concepts, the borderlines of the process can be expanded. For example, the possibilities of integrating sales management into the domain of the commissioner can be evaluated, thus allowing early contributions in the planning phase of infrastructure projects. In the last part of the process, development of possible after-sales management can provide different services in the 'operation and maintenance' stage.

By *product development* is meant a continuous loop of optimisation and refinement of the features of the product, resulting from feedback of deficiencies from the whole process, as well as the initial design stage of new products. The aim of the improvements is to develop the performance of the product, covering a wide range of aspects such as simplified production, simpler assembly or jointing on site, emphasising more efficient construction, but also features such as a more appealing layout of the design or improvements in durability that are more directly relevant to the customer relationship. *Product development* has a very important role in ensuring competitiveness over a period of time; hence the emphasis is mainly on value creation and value management (the V view in the TFV philosophy).

Since an industrial process will involve a different way of working in the design stage, the *product development* implies a new role for the designer as a product developer, rather than a producer of the present unique project-based design. Thus, the designer becomes upgraded to one of the most important persons governing the success of each specific industrial concept in the long run. Additionally, in an industrial process an integrated design is essential, so that production parameters such as constructability and manufacturing aspects continuously can be taken into account.

Productivity development, as a cornerstone in this context, embraces the manufacturing issue of how to attain more efficient and less expensive production, e.g. by rationalising prefabricated production in factories or the execution of assembly on site. Thus, the main emphasis of productivity development is on the transformation vein, the T view of the TFV concept – to rationalise and execute the decomposed transformation tasks in the most efficient way. In a sense, *productivity development* is the ultimate goal, i.e. more efficient construction of bridges, while the two other cornerstones tend to be the means of fulfilling this goal, by developing the products, smoothing the way and providing the necessary tools for governing progress in the right direction. But on the other hand, none of the cornerstones will stand on its own.

Ways of enhancing efficiency in the transformation tasks will differ. A probable feature is an increasing mechanisation of construction, both on-site and off-site.

Further developments of machinery, tools and other equipment sustain the trend of man being replaced by machines. Aided by powerful computer support and ICT, the boosted automation allows industrial manufacturing in construction to use robotics extensively, especially in off-site production. Hence, the production will become more high-tech and the production facilities will be more like any ordinary industry for both on-site and off-site production. This tendency currently seems to be strongest in Japan where several examples of automated industrial building systems can be found, as shown e.g. by Warszawski (1999), but also in Sweden, where the development of industrial housing concepts has been intensive in recent years, as mentioned earlier.

2.2.5 Technical necessities

Technical developments are necessary for industrial bridge construction since, as noted already, more than half of the basis for the process falls in the technical domain. To develop competitive industrial concepts, the essence of recent research in a variety of fields needs to be extracted. The technical necessities are mainly related to the cornerstones of *product development* and *productivity development*.

The importance of environments encouraging innovations must be stressed. It is essential to design the industrial process in a systematic way that enables innovative ideas and improvements, throughout the organisation, to surface.

Furthermore, in Paper II, research areas with high potential have been identified, and priorities between different developments are set. Since it is difficult to judge the priorities for new or emerging technologies, especially when there is little or no previous experience of their performance, an evaluation will have to be somewhat subjective. In the following, brief remarks on some of the most interesting development areas are given.

One of the most important reasons why contemporary concepts of industrial bridge construction have much better prerequisites and possibilities to succeed than their forerunners is the continuous and rapid pace of development in *information and communication technologies* (ICT), along with a growing understanding of how to use and benefit from this computerisation. Support from ICT can be utilised throughout the manufacturing process with databases governing the total process as well as all the sub-processes; compare e.g. Jongeling (2006).

ICT could permit the important integration of design and production to take place, with enhanced planning in all stages and improved control and performance, for example in the 'operation and maintenance' stage. Ultimately, this will allow each construction project to be pre-built and visualised on the computer, thus avoiding or rectifying many deficiencies beforehand, and enabling a differentiation of the production to obtain customised products, i.e. computer-integrated construction (CIC).

Another important area to gain from developments in ICT is the area of quality assurance, closely connected with progress by continuous improvements. ICT can facilitate an enhancement in the feedback from performance in the different fields, and monitor the impacts of the improvement as well as governing the implementation, thus closing the circle of continuous development.

Furthermore, another exciting tendency is the development of smart structures, i.e. structures with embedded systems (e.g. microchips, optical fibres, etc.) which, for example, can communicate to simplify construction or facilitate continuous monitoring and the need for maintenance, or even effectuate changes in structural performance.

The ICT systems can also be used as a management tool, for risk assessment, as supply network tools, in the electronic trade, for educating staff and for feedback, etc.

The field of *materials science* embodies a vast potential to serve as the innovative basis for development of industrial bridge concepts. The high incentive of materials developments is often referred to in the literature, e.g. by Ashby & Jones (1996) who state: 'Innovation in engineering often means the clever use of a new material – new to a particular application but not necessarily (although) new in the sense of recently developed'. Similarly, Li (1993) recognises advanced materials as an increasingly enabling technology for the construction industry. An application made feasible through materials developments is the structural joint between prefabricated elements which is described in Chapter 3. There are many applications that seem interesting in the framework of industrial concept development, and only a few are mentioned here.

Interesting applications in the field of concrete are high-performance concrete or ultra-high-strength concrete, self-compacting concrete, fibre-reinforced concrete (see e.g. Ay (2004), there is a variety of different fibres), lightweight aggregate concrete, and of course combinations of these. Equally interesting materials are high-strength steel and the results of progress in fibre-reinforced polymer composites (FRP). Moreover, techniques such as concrete-filled steel pipes or externally prestressed concrete, and new developments in adhesives and joints between components, seem highly applicable in the context of industrial bridge concepts.

The problem in implementing materials developments is not to discover potential applications, but to overcome the initial stage since new materials tend to be rather expensive before they are commonly used in larger volumes.

The *design issues* are crucial when implementing emerging technical solutions, e.g. based on new materials developments. However, it is difficult to draw the line between materials science and structural engineering, since a closer co-operation is needed between the two, compare e.g. Li (2007). Research in structural design will have to provide rules on how to design with the new materials, which is also essential in the case of several materials being combined, i.e. in composite action. Thus, the research concerns the conventional aspects of design, such as safety against failure, serviceability demands, and fatigue. Additionally, the important aspects of durability and lifetime assessments need attention, e.g. in terms of life-cycle cost and life-cycle analysis, see e.g. Sarja (2002).

A crucial design issue is to find the optimal structural forms for new materials, since the materials might not be efficiently utilised if the conventional forms of ordinary materials are simply copied. Another issue is to find the optimal combination of different materials and to provide a specification of the material properties that is wanted, e.g. for engineered materials.

Further developments in ICT and computerised design seem likely to influence the design process considerably, especially regarding calculations/analyses and drawings (e.g. integrating the two into one transformation task and taking other parameters into account, model-based design etc.), as well as for communication with other participants, e.g. co-ordination with road design in early stages. ICT is also necessary if the fullest sense of computer-integrated construction (CIC) is to be facilitated; thus possibilities to build the bridge beforehand on the computer will further enhance waste reduction and error correction. Furthermore, an enhancement in the use of advanced analysis and simulation methods can be anticipated; e.g. the use of finite element methods (FEM) can become more commonly widespread, possibly in connection with optimisation procedures. In addition, the matter of designing standardised interfaces for multipurpose use, allowing components to be combined interchangeably, seems a key issue for industrial concepts.

Proper detailing has been recognised, e.g. in Paper II and elsewhere in the literature, as one of the most significant areas to develop in order to achieve an efficient construction process. It is often the detailing that governs the ultimate performance of a specific industrial concept, given that other prerequisites are in place. Concerning building systems, a vital issue is to develop fast and simple detailing of joints between components.

Design issues connected with the constraints of production, transportation, erection and assembly, are also important. Production-friendly solutions allowing a rapid and smooth transition through these phases are valuable.

Rational *production methods* are of utmost significance in reaching the goal of substantially increased efficiency. As mentioned, the progress into industrial concepts will yield more high-tech production in environments similar to any ordinary industry, for both on-site and off-site production. Hence, weather-protected production (indoors in industrial facilities, or temporarily covered on-site work) as well as provision of efficient facilities, equipment and safety precautions can become common production features. It is important to remember, though, that it is the supply of the appropriate methods which gives increased efficiency, while weather protection etc. provides the means.

Another probable characteristic of future construction is that machines will replace heavy manual work, for economic reasons (cost reduction) but also due to problems in finding employees willing to undertake such heavy tasks and for labour safety. This trend can already be seen, with demands from authorities on labour environments as the driving force. Hence, there could be a significant increase in mechanisation of construction, both on-site and off-site. The aid of powerful computer support and ICT will promote the speed of this conversion into automation, and the use of robotics in industrial manufacturing can become a common feature in construction, especially for off-site production.

Regarding the completion of structures on site, emerging methods of erection and assembly as well as other on-site preparations will strive towards simplicity with a minimum of manual work and a high degree of mechanisation. Developments in heavy lifting, mounting, jointing etc., will encompass many special machines and equipment to allow rapid installation. Hence, large parts of structures, components as well as systems, are likely to be manufactured elsewhere and assembled on site while stressing the importance of completing the structure at once, thus avoiding later on-site re-establishment for complementary work.

2.3 Efficiency-driven process enhancements

2.3.1 The overall process and sub-processes

It is essential that the overall process is able to capture the specific benefits of industrial construction. It is also important that the process allows continuous feedback between the stages, to be used to develop the process and the products, as can be seen in the suggested process in Figure 4. This process embraces the maximum space that is available for an industrial process; hence, most of the upstream stages are left out. The reason for this is that there are no differences in these early upstream stages of the process compared to the traditional process (which is presented in Paper I), while these are mainly influenced by the public and society.

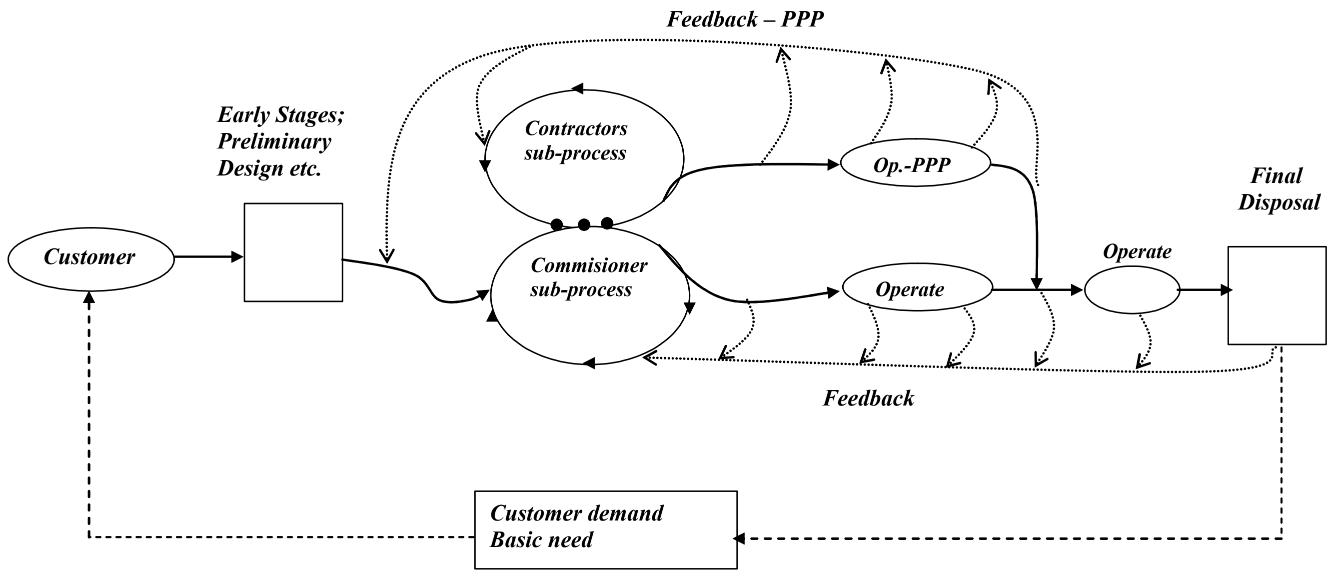


Figure 4. A suggestion of the overall industrial bridge construction process.

In the ‘operation and maintenance’ phase (i.e. Operate or Op. in Figure 4) the suggested process is divided into two parts, where one part encompasses the long-term relations that seem favourable for the industrial ideas, e.g. functional contracts such as different types of public-private-partnership (PPP).

In turn, the process can be broken down into different stages of external or internal sub-processes. The sub-process of a process orientated contractor, for example, could be divided into main processes, supporting processes and management processes. A similar division can be made for the sub-process of the commissioner.

An outline of the contractor’s sub-process from a managerial perspective is shown in Figure 5. The process contains three parallel streams with regard to the TFV theory, concerning the three cornerstones of industrial bridge construction: productivity or production development with main emphasis on the T view, process development with main emphasis on the F view, and product development/design with main emphasis on the V view, with integrated linkage between them. A further development would be to integrate all of the activities into parallel streams.

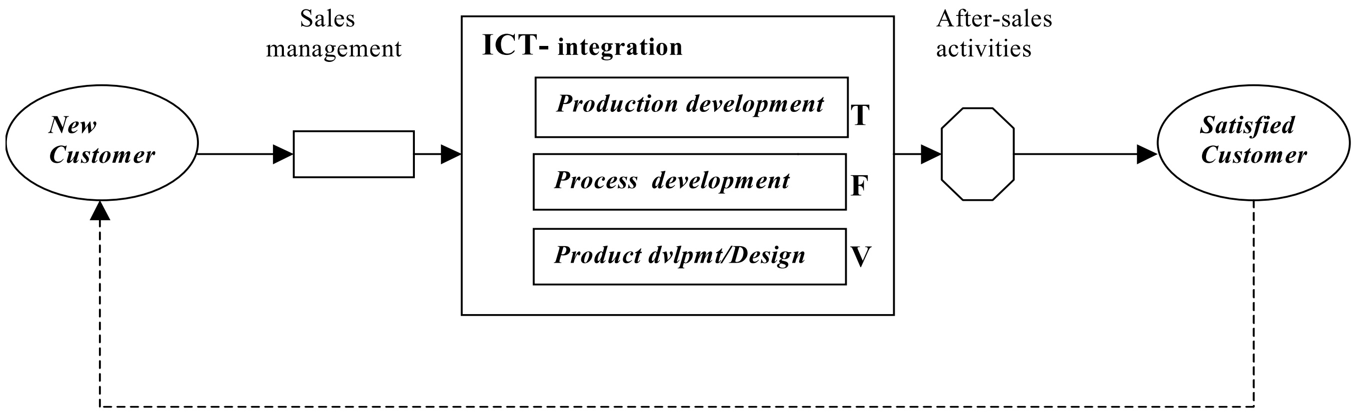


Figure 5. A model of an industrial contractor's process for bridge construction from a managerial perspective.

2.3.2 Efficiency-influencing suggestions

As can be seen from the process flow chart in Figure 4, the sub-processes of the commissioner and the contractor touch upon or slightly overlap each other only during the actual construction phase. This constitutes a problem since the incorporation of these areas is vital to promote the flow through the process. To create a more efficient process, the sub-processes should be more integrated with a larger common area, as visualised in Figure 6. In addition, other areas of the process, such as the 'operation and maintenance' phase, would benefit from being integrated as well. This is especially important for long-term contracts where these parts are included.

Since the procurement phase is the most important phase in terms of motivating the contracted parts, it is envisaged that special attention initially is drawn to this phase. Initially a technical procurement procedure could be used, as indicated in Figure 6, to promote the development of industrial bridge concepts, where innovative technical features are given the same or even higher weight than the economic issues and where life-cycle costs can be taken into account. To evaluate the tenders, a special "technical scorecard" could be used. The scorecard could, for example, be designed similar to the scorecard used for organisational or personal development purposes, but with technical goals instead. In this case it is even more important to clarify the basis for the tender, which is why a special document stipulating the conditions of evaluation should be used. In addition, the form of the tenders should be settled beforehand, e.g. in terms of what functional properties are wanted, etc.

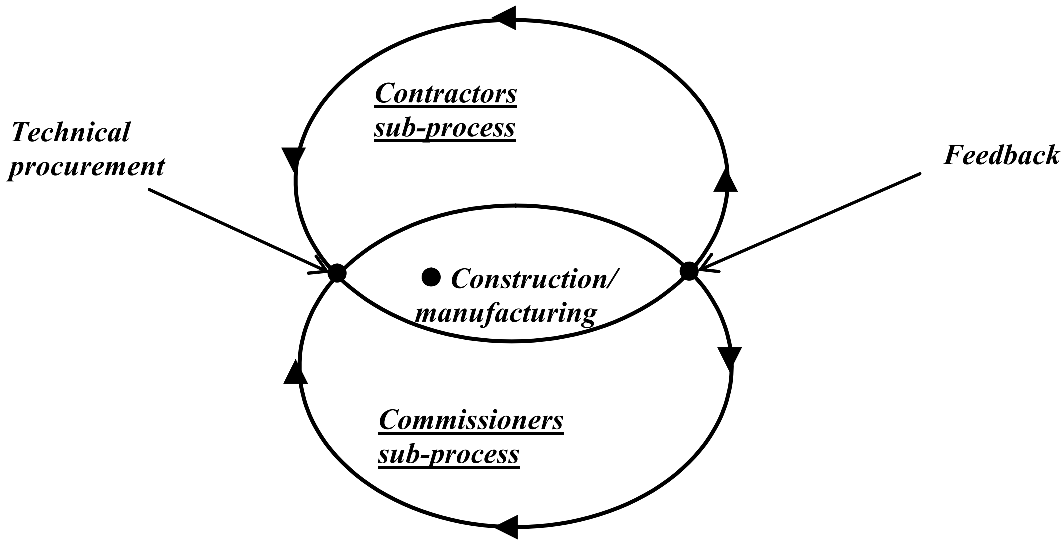


Figure 6. The important integration of the contractors' and the commissioners' sub-processes.

3. Structural developments on component level and detailed level

3.1 Structural developments in general

Structural developments, as indeed every development, are a response to a demand or a need originating from the customer. In conventional construction, it is the need of the specific client in each project that is decisive for initiating developments.

In the case of industrial construction it is mainly the demand for a more efficient production process, originating from the customers, which is the underlying driving force of change. This not only applicable for structural developments, but is also generally relevant for development of the process as a whole including all interrelated areas, as been mentioned. However, from the definition of industrial bridge engineering (see chapter 2) follows the idea of the vital importance of combining structural aspects with industrial aspects in view of improving the efficiency of the industrial construction process. Hence, structural developments as an efficiency-enhancing tool in the progress of the overall industrial bridge construction process are focused upon in this and the succeeding chapter. In addition, it is important to fully integrate the design process into the overall construction process, as reported e.g. by Olofsson (2003).

Structural developments can be performed on different levels in different stages, according to the extent of the improvements as illustrated in Figure 7. The overall level is the system level or the conceptual level, where the general ideas materialise in to physical plans or suggestions and documentations for a specific concept. The next level is the component level, in which the scope gets narrower and where development of the actual components takes place.

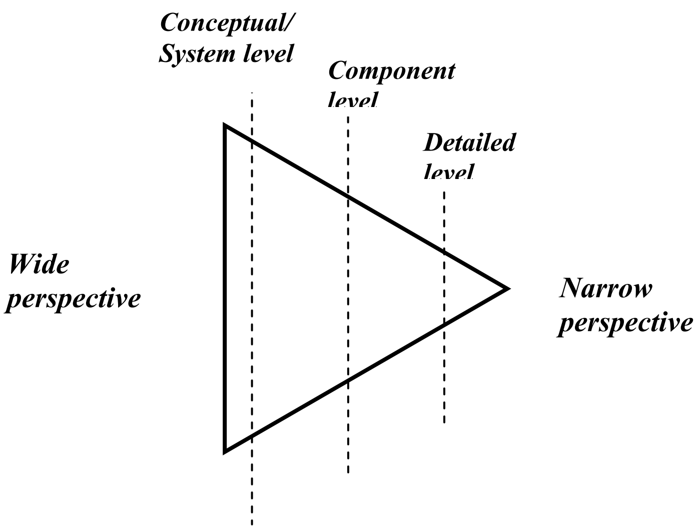


Figure 7. Schematic illustration of structural developments on different levels.

The detailed level is the last level where the details are designed. The development of a new system or concept embraces all of the levels in different stages. The component level can be employed in different cases, e.g. in development of new concepts or for substituting or refining an existing component. The detailed level is always included in new system developments, but can just as well be used within existing concepts, e.g. for rectifying deficiencies or to enhance efficiency in the production process.

3.2 Development of components and details

Structural developments on component level or on detailed level are responds to certain demands or needs, as been mentioned. In conventional construction, it is employed in projects when ordinate solutions are insufficient or cause deficiencies.

In industrial construction it is continuously used to enhance the performance of the construction process and the products. Thus, it is the demands of the market that initiates the developments. It can be used in different contexts, as mentioned, e.g. in developing a system or concept or in the continuous development of existing systems.

An example of structural development on the detailed level in the form of a study of an inventive concept for joint between prefabricated concrete bridge deck elements is given in the continuation of this chapter. The aim of the joint study is to illuminate the possibilities offered by new technology and new materials in developing concepts of industrial bridge construction. It is an attempt to show how structural engineering can enhance the effectiveness of the overall process and encourage development. The study forms a branch in the *product development* part of the industrial construction process. Hence, the focus of the joint study lies in the technical domain of the process, compare Figure 3.

3.3 High-performance joints for concrete bridge applications

3.3.1 Description of the jointing technology

The joint study presented here aims at developing a concept for moment-stiff high-performance joints between prefabricated concrete elements. Several laboratory tests have been conducted which are more thoroughly described in Paper III, as well as finite element (FE) analyses described in Paper IV, showing that the jointed elements can be treated as monolithic members in design. A brief summary of the study is presented here.

The concept consists of wet joints with an ultra-high-performance steel-fibre-reinforced concrete called CRC, Compact Reinforced Composite. The intention is to develop a small joint which is easy and fast to perform, and which makes the surrounding elements continuous, thus rendering the joint especially suitable for use in industrial bridge concepts. The particular joint application in the study is between prefabricated deck slab elements in a composite bridge with steel girders. However, obviously the joint can be widely applied elsewhere in concrete structures. Similar applications for buildings can be found in e.g. Jensen et al. (1996) and Altamimi et al.

(1999). A comparison between one type of conventional joint and the proposed connection is shown in Figure 8 and pictures of the layout can be found in Figure 9.

The CRC concrete is used in order to achieve exceptionally good joint properties. The joint is only 100 mm wide and the arrangement of the spliced reinforcement is uncomplicated, with protruding straight bars and transverse bars simply placed on top. Thus the assembly and casting on site are very simple procedures. Despite the small dimensions, the joint is fully moment-resisting and stronger than the surrounding elements.

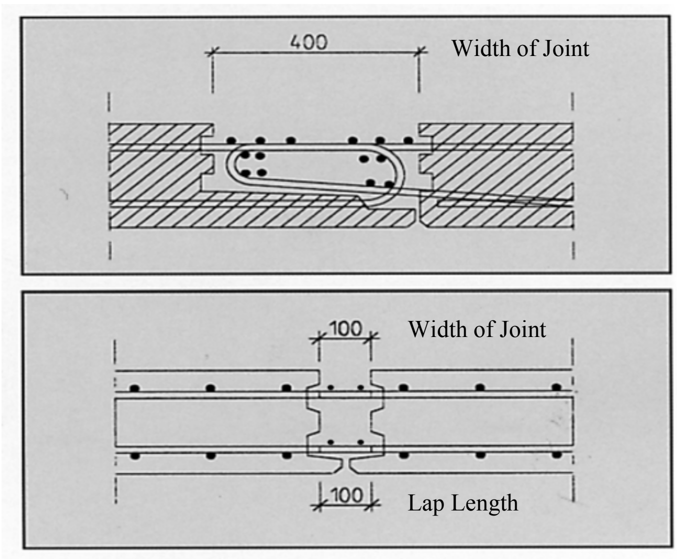


Figure 8. *Comparison of joints between prefabricated deck slabs: a conventional joint (above) and the new joint (below).*

3.3.2 Industrial implications

The field of detailing has been recognised as one of the most important areas to develop in order to enhance efficiency in construction. A major issue is the structural joints between components, as surveyed in e.g. Paper II, especially when prefabricated concrete elements are concerned. The joints are among the key factors that govern the overall performance of construction on site. Moreover, the connections between elements often determine the general performance and structural behaviour of the systems.

Especially for prefabricated concrete, new methods of jointing elements are of crucial concern in order to enhance effectiveness and competitiveness.

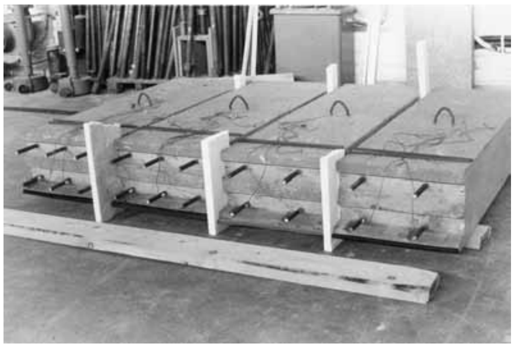
 

Figure 9. At left is the joint before casting, with the reinforcement to be spliced and the transverse bars on top. At the right are four halves of specimens before jointing.

3.3.3 The CRC concrete

CRC is a commercially available product which is silica-fume-based and characterized by tightly packed fine and ultra-fine particles in combination with steel-fibre reinforcement. The water/binder ratio is about 0.16 and the silica fume content is 20–25 %. Quartz sand with particle diameters up to 4 mm is used as aggregate. The compressive strength of CRC is very high, normally about 150 MPa, but higher values can be reached. The fibre content for joint applications is typically about 6 % by volume, or more than 450 kg of steel fibres per m^3 of concrete, which was the amount for concrete used in the joints. These features provide CRC with exceptional properties of bond to reinforcement and, in addition, with good adhesion to previously cast concrete. In e.g. Jensen (1999), Nielsen (1995:I) and Nielsen (1995:II) descriptions of the material and its performance as well as further examples of applications can be found. Because of the low water-to-binder ratio there are moderate shrinkage effects, and for the test specimens these effects are negligible. However, the effects need to be further evaluated for each specific application when CRC is cast on-site, e.g. for the composite bridge concept with steel girders.

A broad documentation of material properties such as durability, fire resistance, fatigue etc. has been supplied for CRC; see e.g. Jensen et al. (1996) and Aarup et al. (1997).

Due to the small dimensions of the joint, only small amounts of CRC are needed. Hence a specially prepared dry mortar mix is used, which means that water is the only additive on-site, allowing very simple mixing of the concrete. Execution of the casting is also simplified; the CRC fills the joint without difficulty and levels smoothly, allowing the joint to be cast by conventional methods with a good result. No compaction difficulties were noticed for the CRC when casting the test specimens, but this issue needs attention when casting the joint on site. As for all steel-fibre-reinforced concrete, possible surface corrosion due the steel fibres should also be mentioned.

3.3.4 Laboratory tests

The laboratory tests performed include static tests of flexural bending moments and shear forces as well as fatigue tests, conducted for a broad variation of the joint geometry in order to find the best solution. The tests have been thoroughly described in Paper III. Hence, only a very concise review is presented here.

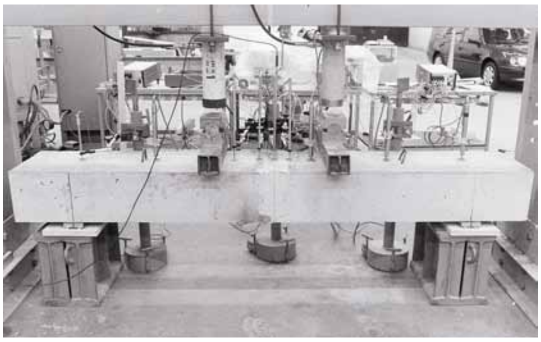
Static tests

The tests have been done in three stages. First, static tests were executed for both shear and flexure capacity, and they gave very satisfactory results regarding shear, while the results for bending were not acceptable due to anchorage pullout failures before reaching the ultimate strength of the rebar. The most important reason for this premature failure was that not enough CRC cover was provided for the rebars in the joints.
Hence, the second stage of testing, also under static conditions, aimed at optimizing the joint geometry with respect to flexural bending capacity, emphasizing two crucial factors for avoiding pullouts: providing a sufficient concrete cover of CRC, and adding transverse bars simply placed on top of bottom rebars to counteract effects of tensile splitting due to bond action. The height of the tested specimens was 260 mm, the width was 440 mm, and two 16 mm bottom rebars were spliced in the joint. The results and a specification of the tested specimens are presented in Table 3, while the set-up of the 4-point bending tests and the loading arrangement are shown in Figure 10. Load versus mid-span deflection for all specimens in the static test can be seen in Figure 11.

Table3. Results from the static tests in stage 2.

Specimen No.	Width of joint (mm)	Cover of CRC (mm)	Transverse Bars	Crack load (kN)	Yield load (kN)	Maximum load (kN)	Deflection at max load (mm)	Type of failure
1	100	20	ø8	19.2	59.5	79.8	47.7	B
2	100	20	ø10	19.9	61.8	84.9	47.9	B
3	80	20	ø8	18.4	61.8	81.6	41.3	B*/
4	100	15	ø8	20.0	60.0	82.9	49.1	B
5	100	15	ø8	19.3	61.3	83.9	45.0	B
6	80	20	ø8	13.5	63.8	78.6	30.5	A
7	100	20	None	18.7	61.9	77.9	30.7	A
8	100	20	None	18.0	61.6	76.7	29.0	A
9	100	20	ø8	17.9	59.8	82.9	51.6	B
10	100	20	ø10	18.1	61.0	86.9	51.6	B

Notes: Lap length is the same as the width of joint. The given load is one of the point loads (the values must be doubled for the total load). Bending failure is specified by B (*/ means a less ductile behaviour in the post-peak range due to insufficient bond), and anchorage failure (pullout) is specified by A.



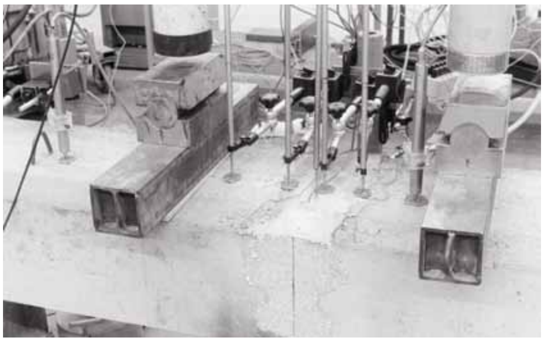


Figure 10. At left is the set-up for the static loading test. At right is a close view of the set-up. Vertical deflection was gauged by displacement transducers and strain gauges were attached to the reinforcing bars to be spliced. Essentially the same set-up was used for two of the series in the fatigue test.

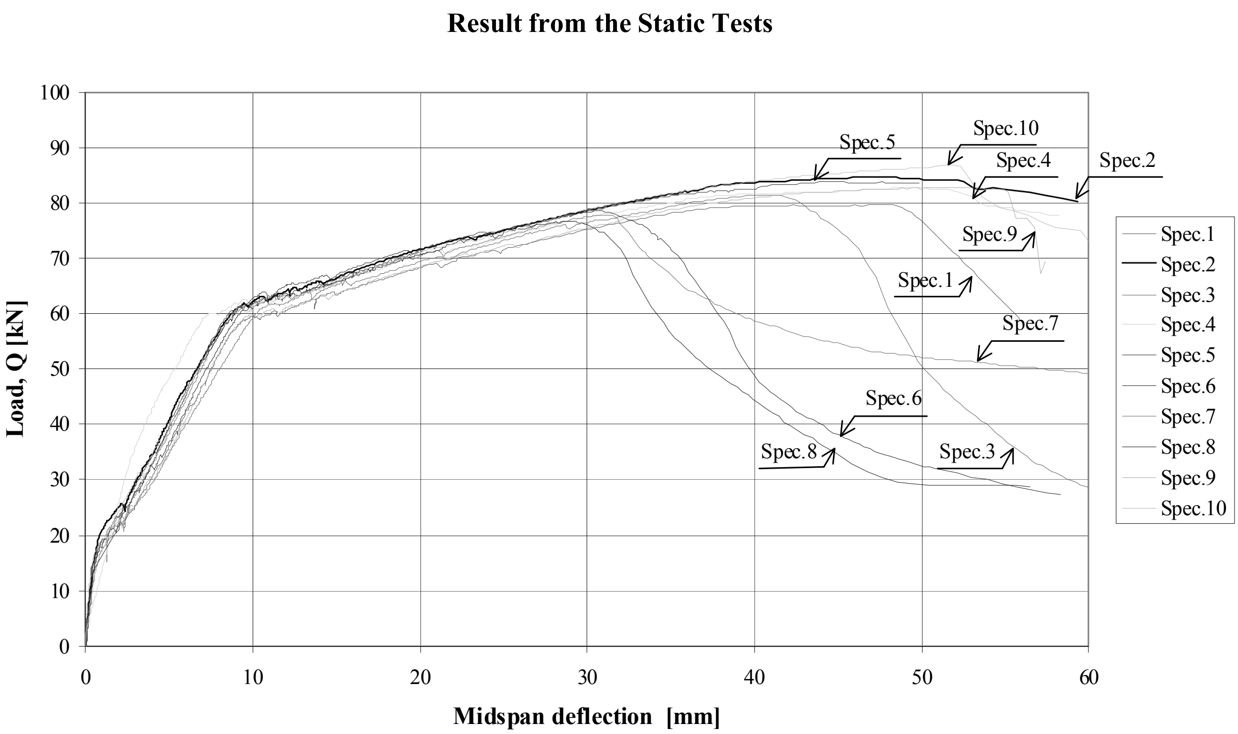


Figure 11. Load versus mid-span deflection for all specimens in the static tests.

All flexure failures took place outside the connection and there were no visible cracks within the CRC in the joint. All failures occurred after yielding of the reinforcement, and most specimens achieved an ordinary bending failure with crushing of the concrete in the compression zone. It was concluded that the presence of a transverse bar was crucial for ductile behaviour.

Fatigue tests

The third stage of tests was carried out to investigate the performance under fatigue. The optimized joint geometry inferred from the static test was used in these test series, featuring a lap length and width of joint of 100 mm, two 8 mm transverse rebars in the joint, and 20 mm concrete cover of CRC. Two of the fatigue test series, UP1 (two ø16 mm bottom rebars) and UP2 (three ø16 mm bottom rebars), were performed on similar specimens as in the static tests (compare Figure 10), the goal being to withstand more than 400,000 load cycles. A maximum stress believed to be substantially higher than the actual maximum stress levels for fatigue in bridge slabs was chosen, about 300 MPa. The minimum load was then calculated to achieve a theoretical stress range of 250 MPa in the rebars.

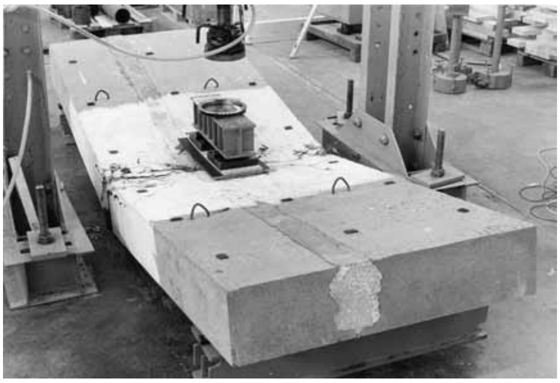
Specimens in series UP3 were oriented in the transverse direction, relative to the other series (i.e. in the same direction as the actual deck slab spans the distance between the bridge beams), with the joint along the elements, as can be seen in Figure 12. The height and width of these specimens were 260 mm and 1200 mm respectively. Additionally, the joint was placed in an eccentric position for these specimens, in order to achieve shear load transfer in the joint. The maximum load was set to135 kN, distributed over a loading area of 200 x 600 mm^2 representing a wheel's pressure. The distance between the supports was set to 2880 mm, chosen in combination with the minimum load to reach a reinforcement stress range approximately at the same level as for the two other test series.

All specimens sustaining more than 800,000 load cycles were loaded statically to failure. As the load cycles continued, the number and width of cracks increased, although no cracks were observed in the CRC joint. Results of the fatigue test series are shown in Table 4. The fatigue failure occurred in the prefabricated concrete elements for all specimens but UP2:2, which failed in anchorage for one corner rebar due to insufficient CRC cover for this bar because of a misalignment.

Table 4. Results from the fatigue tests.

Type of specimen	**Specimen No.**	**Failure type**	**No. of load cycles**	**Crack load (kN)**	**Yield load (kN)**	**Failure load (kN)**	**Deflection at max load (mm)**
UP1	UP1:1	F/B	423000	22	--	--	--
	UP1:2	F/B	506000	20	--	--	--
UP2	UP2:1	S/B	820000	--	201	249	43,4
	UP2:2	F/A	382000	30	--	--	--
UP3	UP3:1	S/B	802000	69	318	373	49,9
	UP3:2	S/B	800000	80	322	385	56,3

Notations: F/B indicates fatigue failure in bending, F/A specifies fatigue failure in anchorage and index, and S/B indicates failure under static loading after reaching more than 800,000 load cycles (deflection and load, other than crack load, can only be shown for these specimens). The given load is the total load on the specimens.



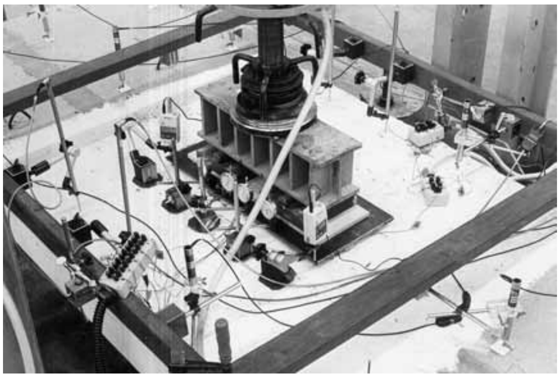


Figure 12. At left is one of the specimens of fatigue test series UP3, after ultimate static loading to failure. At right is a close view of the set-up for the same series. The eccentric longitudinal joint and the centric loading simulating a wheel's pressure can be seen in the pictures.

3.3.5 Numerical analyses

Finite element analysis in two dimensions

The structural behaviour of the joint was analysed by using the finite element (FE) program DIANA (see TNO (1998)), with a plane stress model in two dimensions based on non-linear fracture mechanics. Since the analyses have been comprehensively presented in Paper IV, only a very brief outline is included here. The mesh of the 2D FE model is shown in Figure 13 and the results from the analyses can be seen in Figure 14. The agreement between the curves from the FE analyses and the tests is satisfactory, as can be seen in Figure 14. The difference in ductility between the two specimens is obvious both from the test results and from the analysis. The results from the analyses indicate that the bond capacity of the joint without transverse rebar is about 75 % of the capacity with the rebar present in the joint.

In addition, the specific issue of divergence between the theoretically calculated minimum rebar stress and the actually measured stress in the tests (the latter being significantly higher than the former), noticed from the fatigue tests, was later verified by means of FE analyses. Although on a smaller scale, the phenomenon when unloading the specimens could be compared to the case of anchorage regions at beam-ends for post-tensioned concrete, or referred to as a "reversed" tension stiffening effect.

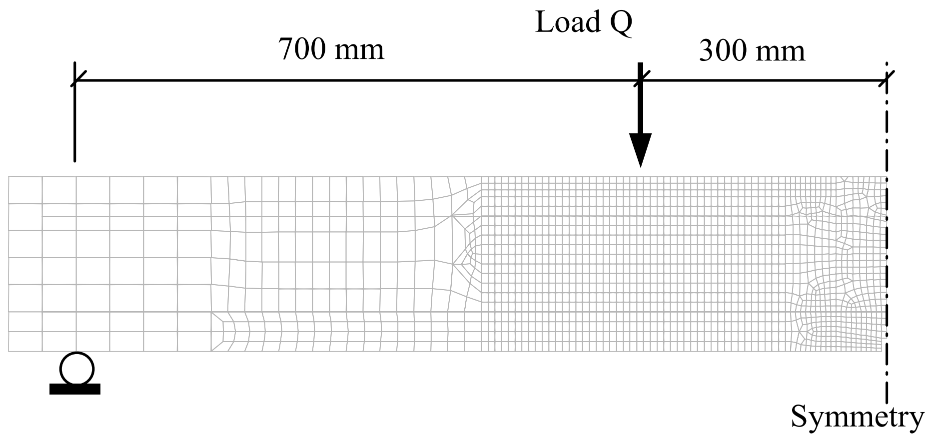


Figure 13. The mesh of the 2D FE model.

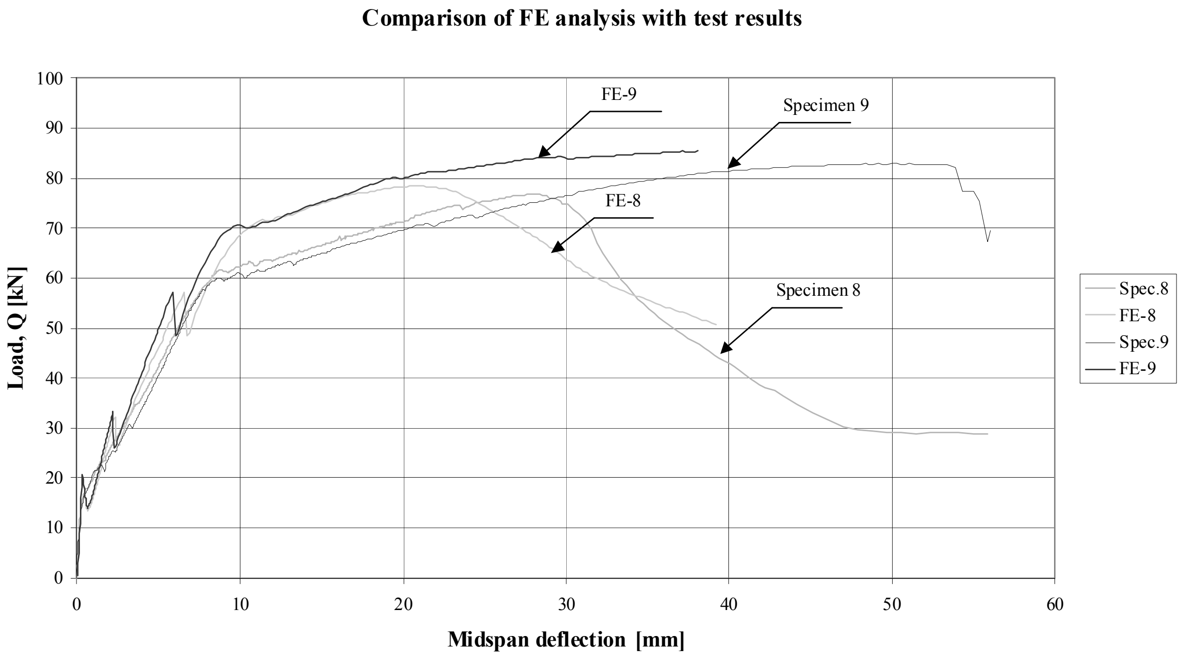


Figure 14. Comparison of 2D FE analysis with test results for specimens with (specimen 9) and without (specimen 8) transverse rebars in the joint.

Finite element analyses in three dimensions

Recent numerical analyses in three dimensions (3D) have been carried out to further evaluate the structural behaviour of the test specimens. The aim of these finite element analyses has been to create a 3D FE model of the joint where the mechanical behaviour in the transverse direction are captured, i.e. also modelling concrete and reinforcement in the transverse direction. The model can then be used for further evaluations, e.g. complementary parametric studies, etc. The analyses were done to complement the previous 2D analyses in order to overcome the shortcomings of the two-dimensional FE model.

The analyses were conducted by using the general finite element program DIANA; see TNO (2005). A three dimensional model based on non-linear fracture mechanics was developed with solid elements. The rotating crack approach based on smeared cracking and total strain was used to model cracking of the concrete and the plasticity model of Thorenfeldt accounted for the non-linearity of concrete in compression. For the reinforcing steel, use was made of the von Mises failure criteria with strain-hardening. Interface elements with multi-linear relationships for the bond-slip were modelled between the concrete and reinforcement elements to capture the tensile behaviour for the bottom reinforcing bars, while for the bars at the upper edge the assumption of full interaction between concrete and reinforcing steel was made and the bars were modelled through the 'embedded' option in the FE program. In addition, the tensioned protruding bars (at the bottom edge) were stiffly connected with the fixed-tyings feature in DIANA to capture the splicing. Due to the symmetric conditions, only one fourth of the specimen needed to be modelled, and the size of the model could be reduced accordingly. Hence, a denser mesh could be used in the parts of the model representing the joint and its vicinity, while a considerably less dense mesh was used in the remote parts of the model, as can be seen in Figure 15.

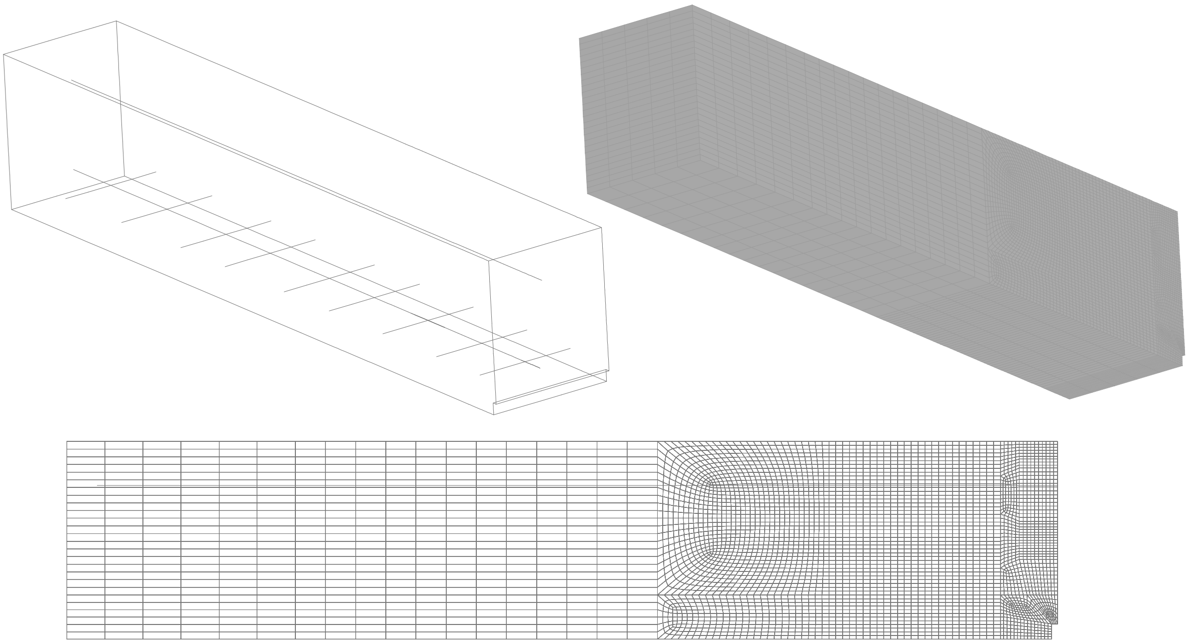


Figure 15. The mesh of the 3D FE model, at top left an outline with the rebars, at top right the "hidden sides" and at bottom a side view. Load and supports etc. are the same as in Figure 13.

Since the only material parameters measured at the time of testing were the compression strengths for the ordinary concrete and the CRC, the additional material parameters were chosen as recommended in the literature and similar to the previous 2D analyses. For the normal concrete the fracture energy, tensile strength, and bond-slip behaviour of the reinforcement were all selected in accordance with Model Code 1990, see CEB (1993). In the model for the bond-slip, 'good bond conditions' but 'unconfined concrete' were presumed. Due to experience with the initial 2D analyses, an empirical value for Young's modulus was selected, since values from the literature led to a much too stiff behaviour in these analyses. Similarly, the stress-strain relationship in the post-peak part was made less step in order to avoid localization of concrete in compression. Beside the measured compressive strength for the CRC, other material properties were evaluated from Heshe (1988), Aarup & Nepper-Christiansen (1992) and Jensen (1995:I), while an approximate bond-slip relationship was derived from results of pull-out tests in Aarup & Jensen (1997). A summary of material properties used in the analyses is shown in Table 5.

Table 5. Material properties used in the FE analyses.

Material	Type	f_{ccm} , f_{sv} (MPa)	f_{ctm} , f_{su} (MPa)	E_{cm} , E_s (GPa)	G_f (Nm/m^2)
Concrete	C60	56,9	4,0	30,0	101
CRC	-	149,9	10,0	48,7	18000
Reinforcement	K500	564	662	210	-

Analyses were conducted both with and without the transverse rebar in the joint, to confirm that it was possible to differentiate between the two cases. Results from the two analyses for load versus mid-span deflection are shown in Figure 16, where difference in ductility can be noticed. However, the curves are somewhat angular since the iterative algorithm for the load steps used rather large steps at high load levels. The algorithm was used due to convergence reasons. In addition, the maximum loads from the analyses were too high which might also be a consequence of the used load step procedure. Hence, it is difficult to compare the results from the analyses with those from the tests and from the 2D analyses.

It could be concluded however that it can be possible to distinguish between the two cases, but further analyses need to be carried out to reach a better agreement with the previous results. Furthermore, although the 3D model is able to represent the transverse reinforcement, the material model for bond-slip behaviour is still unable to represent the splitting stresses which occur in combination with the bond stresses for ribbed bars. Additionally, the reduction of bond stress when yielding of reinforcement occurs cannot be captured by the material model either. Hence, it could be wise to use a bond model capable of describing these actions, for example the bond model developed by Lundgren (1999), in the case of further analyses. This would allow a generalisation of the results from the laboratory tests in e.g. parametric studies, to thoroughly evaluate the geometric sensitivity seen in the tests, compare Paper III. In addition, more decisive conclusions about guidelines for design as well as for production (e.g. tolerances, etc.) could be drawn. Moreover, it would provide enhanced knowledge about the mechanical behaviour of the joint and especially about the influence of the transverse rebar.

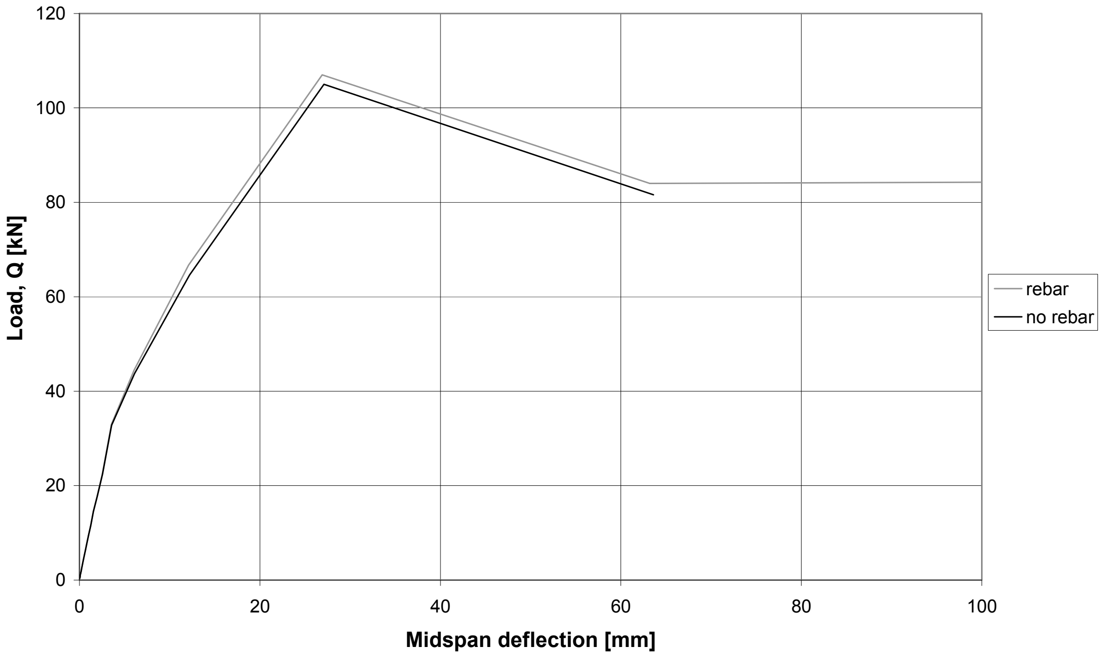


Figure 16. Results from the two FE analyses – with and without transverse rebar in the joint – for load versus mid-span deflection.

3.3.6 Concluding remarks

The main result from the study of the high-performance joint concept is that the joint is stronger than the adjacent prefabricated elements if a proper detailing and a sufficient lap length are provided. A minimum lap length of 100 mm was found to be adequate if two transverse reinforcing bars were simply put on top of the spliced bars. In such cases, continuous structural elements treated as a monolithic member in design can be created of precast concrete elements connected by this joint.

Additionally, the geometric sensitivity that was seen in the laboratory tests, compare Paper III, ought to be further investigated, e.g. by means of a parametric study using 3D FE analyses, before any decisive recommendations about guidelines for design as well as for production (e.g. tolerances) is given.

4. Structural developments on system level or conceptual level

4.1 Conceptual and system developments

Developments on system level or conceptual developments, are the result of certain demands or needs as mentioned in the previous chapter. In conventional construction it is the first phase in the design process where the demands of the client are transformed into a project.

In industrial construction it is the foundation for the industrial developments to be undertaken in order to produce and implement the product on the market. Hence, it is the market demands that govern the conceptual developments. It can be the first stage in developing a new concept or system, resulting in new products, but it can also be the continuous development of existing concepts.

An example of structural development on the conceptual level in the form of a feasibility study for a new bridge concept is given in this chapter.

4.2 Feasibility of bridge prototypes

It was decided in an early stage of the research project that a feasibility study of a potentially successful industrial bridge concept was to be undertaken, including a conceptual design phase. The aim of the study was to elucidate the possibilities offered by new technology, new materials and other advancements when developing concepts of industrial construction (i-construction), and especially industrial bridge construction concepts. Hence, structural engineering and design aspects were focused upon, along with industrial aspects and other critical issues as stated in the following sections. The study forms a branch in the *product development* sub-process of the industrial construction process (compare Figure 3). Still, the contribution to the overall process can be significant even though this is not the focus of the study. The study constitutes an attempt to demonstrate how new developments can be utilised in bridge construction and how structural engineering can contribute to enhance the effectiveness of the overall construction process and encourage development. Thus, the main focus of the study lies in the technical domain of the process, dealing mainly with technical assessment of structural elements. The studied bridge may perhaps never be built in a larger scale, but parts of the concept can still be interesting in other cases. There are certain similarities to how the car industry develops concept cars which maybe never are produced in more than one prototype, but different developments from the concept will be incorporated into the new car models to come.

There is a variety of demands and requirements to consider when developing a novel bridge concept. In addition, there are many desired kinds of performance with added value that would be beneficial to achieve. One task of the feasibility study is to illuminate all crucial concerns that could obstruct the concept from being realised. Reasonable validation of these matters must be provided to confirm the feasibility of the concept. A wide range of questions need to be answered. Subjects range from production matters, assembly, erection, industrial matters, economics, structural aspects, design, aesthetics, environmental issues and durability to time of construction and efficiency. To generalise, the issues are divided into two parts. The first general part mainly concerns industrial aspects and the second part concerns structural matters and design.

In addition, there is a variety of tests that are necessary and beneficial when developing bridge concepts. To validate the structural performance, one needs material tests as well as tests of details, but also tests of the whole structure or large parts thereof. Material parameters as well as design requirements are needed and some crucial details must be investigated. In feasibility studies, however, the areas of interest must be limited to the most essential issues. It is important to remember, though, that the investigations which are conducted in a feasibility study are initial ones, and that refinement is of course necessary in a later stage.

Since it also was an aim to try to utilise new, or in construction not commonly used materials and developments, there was a risk that the concept would not be economically competitive in the current market, but this was to be disregarded. If the concept does not seem to be feasible today for economic reasons, it should still possess a large future potential. Obviously, due to the nature of a feasibility study the bridge only exists in theory as yet, and what is said about the concept at this stage consists of well-grounded but fictitious assumptions.

The study is divided into three parts and a constrict summary is presented here, while a more comprehensive description can be found in the appended papers. The first part of the study, referring to Paper V, is presented in section 4.3 and it is dedicated to a general description of the concept as well as the initial investigations and numerical analysis conducted. The second part presented section 4.4 and referring to Paper VI, concerns an experimental study of bonded interfaces in the bridge deck including laboratory tests. In section 4.5, the third and last part can be found, which deals with the assessment of a prototype test beam and covers both laboratory testing and finite element analyses. It refers to Paper VII.

4.3 The feasibility study of the *i-bridge* concept

4.3.1 Prerequisites and preferences

Certain requirements have been stipulated for the feasibility study, such as that the concept must fit into an industrial process and that industrial methods and developments were to be utilised.

The aim of industrial construction, and indeed for any industrial process, is to make products at a lower cost or alternatively to make products of higher quality at the same cost. The optimum is of course for both criteria to be fulfilled at the same time, i.e. make products of higher quality at lower cost.

A summary of the desired requirements for the bridge concept is presented in Table 6.

Table 6. Desired requirements for the bridge concept.

Basic Requirements

- functionality, safety, serviceability and comfort criteria
- manufacturing suited for an industrial construction process
- sustainability, durability, maintenance and repair aspects taken into account
- quality aspects, lifetime assessment and environmental concern.
- economic, flexible and efficient construction
- short time of construction (on site)
- labour-friendly working environment

Requirements causing enhanced customer value

- substantial reduction of waste, reducing overall construction time and cost
- simple and rational construction or manufacturing, resulting in lowered construction cost
- better use of resources from a public economic point of view
- minimisation of life cycle cost and environmental effects
- improved quality aiming at zero mistakes and consequently no guarantee work
- continuous development and improvement of processes, products and productivity
- flexible process, both during construction and afterwards without disturbance due to construction work
- developed ICT systems to govern the overall process, resulting in e.g. better planning, easier procurement and less administration.
- optimisation, easy, fast and straightforward design, integrated in the overall process
- enhanced durability and reduced need of maintenance and repair; if still needed, it is to be pre-planned, easy and fast to execute
- simple and minimised work on site, and all work carried out in sequence, thus avoiding later rework
- simple and light elements to minimise transportation, avoid heavy lifts etc. and to provide efficient assembly
- a high degree of prefabrication, preferably with no on-site work except erection and assembly
- enhanced flexibility with the possibility to adopt to different road designs
- possibility to alter the aesthetics and the appearance of the bridge
- possibility to complete the bridge in one week on site with only short interruptions in traffic (when built over existing roads)
- non-labour-intensive production and erection

Special requirement for the study

- to utilise and demonstrate new or not often used technical developments

Many developments in materials have not become commonly utilised in construction, out of different reasons. One group of materials still awaiting their large-scale breakthrough in construction is fibre reinforced polymers (FRP). Great effort has been put into research and development of these materials in order to enhance their utilisation in construction. Until today, however, they are mostly used in special applications and not generally widespread with exception for strengthening of structures where the use of FRP is the state-of-the-art. The large advantages of FRP, as stated e.g. by Keller (2003), are their excellent specific strength and that they can easily be formed into any shape, are largely corrosion-free and are largely resistant to fatigue. Structural components of FRP can be industrially produced and can be erected on the construction site in a very short time without the need of heavy lifting equipment.

Another material that has been emerging for quite a while is ultra-high-performance steel-fibre reinforced concrete (UHPSFRC). This is a material that has been commercially available for more than a decade, but still has not been utilised on a large scale and is currently used mostly in special applications. It is able to cope with large compression and to work in composite action with e.g. FRP. The tension capacity is preferably large enough to avoid the use of reinforcing bars. It fulfils the criteria of a robust, dense and rough material that is needed for the wearing surface of the deck, if the deck is to be durable even without protection from isolation.

Hence, the choice of materials for the concept fell upon FRP in composite action with ultra-high-performance steel-fibre reinforced concrete, as been indicated above. The materials are glass fibre reinforced polymers (GFRP), carbon fibre reinforced polymer (CFRP) and CRC concrete, where the FRP are used in the major load-bearing components and the concrete is utilised in the deck. Among the main reasons for this choice were durability concerns, since there was a wish to exclude the use of reinforcing steel and to utilise more maintenance-free materials. It also seemed challenging to provide for composite action between the different materials, similar to the research done to provide for composite action between steel plates and concrete; see e.g. Walter (2005).

For the GFRP, E-glass fibres in a vinyl ester resin were chosen. The CFRP consists of carbon fibres of intermediate modulus (IM) in an epoxy matrix. The CRC concrete, Compact Reinforced Composite, has been presented in Chapter 3.

The *i-bridge* concept should of course allow efficient industrial production of the bridge with large flexibility and variation, to be able to adapt to different situations and locations. The general idea is to combine and utilise appropriate materials and to exploit their entire capacity. Hence, the bridge type and the geometry should be appropriately chosen in order to fully utilise the performance of the materials. The concept in the feasibility study refers to road bridges of normal span length, while the shape of the load-bearing components was received in the conceptual design on the basis of the most important issues focused upon, i.e. efficiency and industrial matters.

Since both FRP and ultra-high-performance steel-fibre reinforced concrete are materials enabling a choice of tailored material parameters, i.e. engineered material, the main task for the designer is to specify the parameters to suit the design – rather than adapting a design to meet predetermined material performance, which is the common approach to design in construction.

Obviously, it is not possible to cover all part of the bridge development and design in detail. Since the aim of the feasibility study is to determine whether the bridge concept is possible from industrial and structural engineering points of view, the main focus in the study lies on the industrial issues and the conceptual design phase, e.g. to ensure that the overall static system is working properly.

Thus, there are many details that are left out or not covered in depth. However, there has been an intention to adopt a holistic view in the study. Efforts have been made at least to mention all relevant issues and try to present plausible solutions, although the investigations needed to ensure their correctness are lacking.

Aspects covered more in depth in the study are the conceptual and preliminary structural design of the main load-bearing parts of the bridge, i.e. the bridge beams and the deck of the superstructure. No optimisation of the structure has been done, though. In addition, only one-span bridges are considered at this stage.

4.3.2 Outline of the bridge concept

The superstructure of the *i-bridge* concept consists of v-shaped GFRP beams reinforced by CFRP profiles, with a deck consisting of GFRP plates in composite action with UHPSFRC. The CFRP profile is placed inside the beams, mainly for aesthetic and protective reasons. The concept is outlined in Figure 17–19. Ideas for foundation and substructure is summarised in Table 7. Principles for a suggested foundation with reinforced earth are shown in Figure 20.

The bridge in the study is freely supported, while multi-span continuous bridges are excluded at this stage. The span width designed for in the study is 25 m, which is a common intermediate span that can accommodate a magnitude of bridging situations.

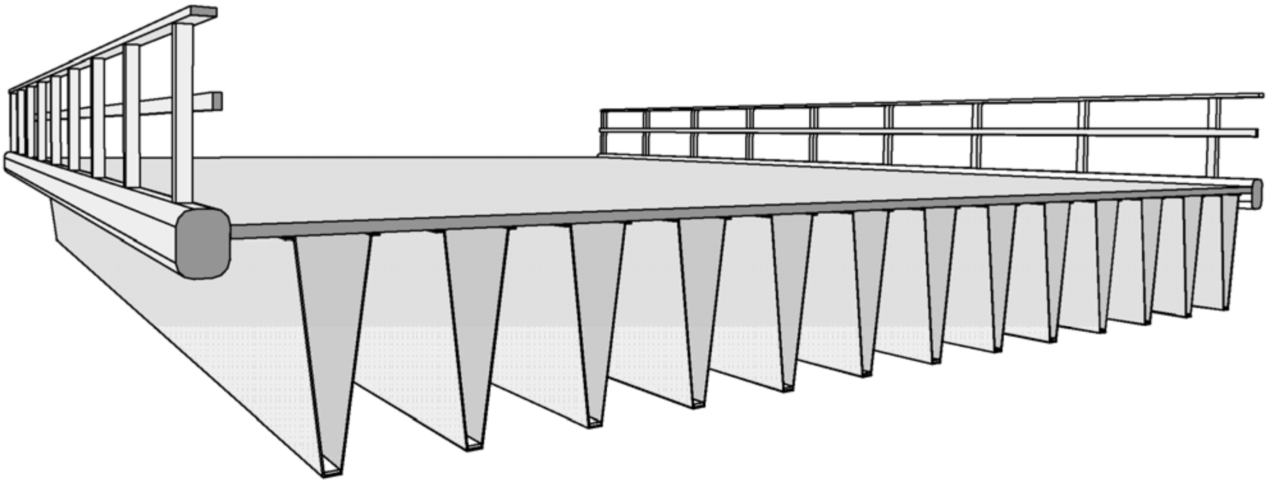


Figure 17. Outline of the i-bridge concept, a perspective sketch illustrating the major components of the bridge.

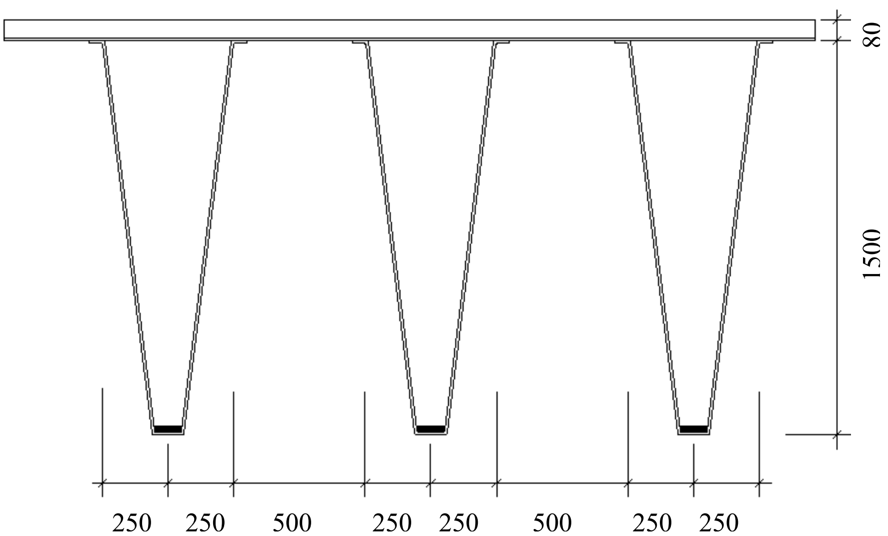


Figure 18. Major dimensions of the cross section (in mm).

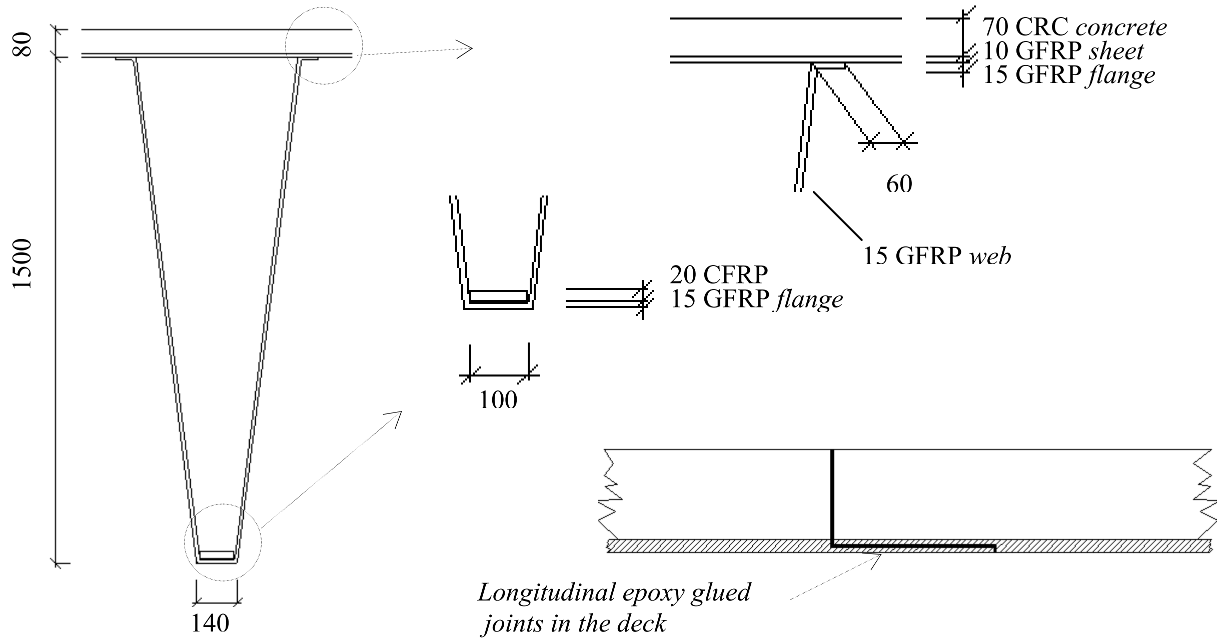


Figure 19. Detailed dimensions (in mm).

Table 7. Summary of ideas for foundation and substructure.

Groundwork and foundation	***Substructure***
- easy and rapid to perform	- simply consists of a slab on top of the reinforced earth
- substantial savings can be achieved due to the low dead weight of the bridge	- in case of very good ground conditions, the reinforced earth can be excluded
- foundation by means of horizontally reinforced earth is possible	- reinforced earth also provides the appropriate bridge level without the need of abutment walls
- generally not necessary to strengthen the earth vertically	- dimensions of wing walls and end walls are minimised so that they can be prefabricated and mounted on site
- in the case of worse soil conditions, the lime column method will most often be sufficient	- a run-on slab secures the elimination of differential settlements, where needed
- conventional piling is only needed for very bad soil conditions, e.g. for soft clay	- reinforced earth employed also behind the bridge means that wing and end walls can be excluded
- the reinforced earth can be complemented with a lining depending on prerequisites in each location	

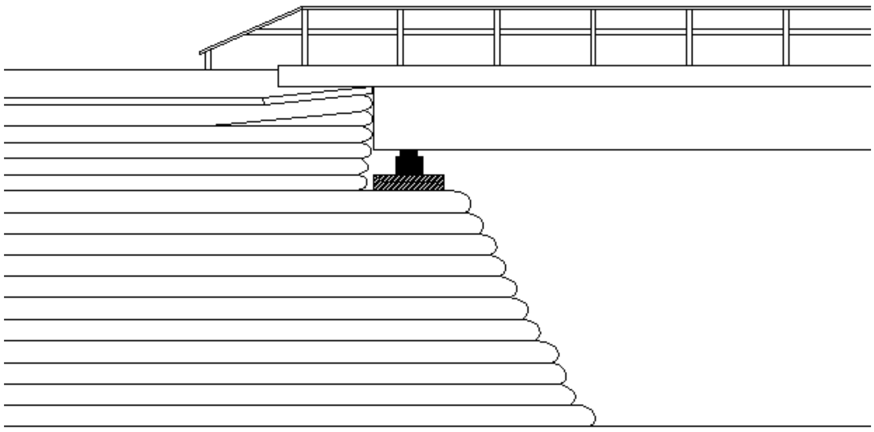


Figure 20. Elevation showing principles for the proposed foundation and substructure of the bridge.

There are huge possibilities to include different kinds of monitoring equipment when manufacturing the different parts, e.g. fibre optic sensors (FOS). Hence, the bridge performance along with other parameters, e.g. traffic loading, axle loads etc., can be remotely monitored by the operators. In addition, transmitters can be included in the structure to create a "smart structure", e.g. to facilitate the assembly and erection.

4.3.3 Industrial features

The nature of a feasibility study means that the bridge currently only exists in theory, as been mentioned. Hence, a vision of how the industrial features and the construction process for the *i-bridge* concept could look is reviewed in Table 8.

Table 8. A vision of the industrial features and the construction process for the i-bridge concept.

Industrial characteristics

Production in general

- all parts of the bridge are manufactured indoors in factories and transported to the site for assembly and erection
- all production in the factories can be automated to a large extent
- the factories manufacture on demand and the elements are produced according to the just-in-time philosophy, eliminating the need to keep components in stock
- lean production is adopted and thus the flows through the factories are facilitated in order to eliminate congestion and reduce storage
- elements are light and easy to transport
- a few trucks are generally enough to transport a normal-sized bridge to the site of erection
- the rapidly performed on-site work is reduced to erection and assembly, besides the substructure
- a small or normal-sized crane is sufficient to perform all lifts for completing the assembly
- the bridge are ready to use a few days after erection

The use of ICT

- a database governs the total process as well as all the sub-processes
- the erection is supervised by GPS via cast-in computer chips, ensuring correct positioning of each element
- design and construction are fully integrated
- all bridges are pre-built in the computer in order to check procedures and correct mistakes
- computer-integrated construction (CIC) is used
- the bridge is designed from data bases where data are reused
- the ICT systems are also used for production planning, as a management tool, for risk assessment, for quality control, as supply network tool, in the electronic trade, for continuous improvement, for educating the staff and for feedback

Flexibility

- there is a huge flexibility of the concept
- the production can easily be altered to adjust to specific situations
- easy to change parameters in production, material properties etc.
- easy to change the span and the cross-section of the bridge
- concurrent engineering is adopted, while flexibility also is easily supported in design
- material requirements are altered to the specific case before being sent to the shop for fabrication, resulting in high material utilisation and a significant reduction of waste

Manufacturing

- the GFRP beams and the CFRP profiles are manufactured through the pultrusion process
- after the beams have been cut into the appropriate lengths, the CFRP profile is glued to the beam with an epoxy adhesive
- the GFRP plate is manufactured by the vacuum bag process with a special rough surface
- the deck is prefabricated and the concrete is cast upon the GFRP plate at the factory
- the deck is produced in approximately 3 m wide elements with the same length as the full width of the bridge
- the deck is jointed to the upper flanges of the beams by epoxy adhesive
- crossbeams are installed at the supports to stiffen the beams for the concentrated reactions
- the edge beams are designed to distribute the collision force from the parapet into the deck
- the edge beam is made of GFRP and concrete in composite action, and it can be prefabricated as a full-length element or it can be included in the deck elements

Assembly

- the assembly is characterised by a stepwise erection in just a few phases
- when the foundation and the substructure are completed, the beams are placed on rubber bearings on the seatings by a mobile crane
- the elements do not weigh more than about 6 tons, so a normal crane will be sufficient
- in favourable conditions, the erection can even be done directly from trucks equipped with cranes
- after the placement of temporary bracings, the deck is put in place after the epoxy has been spread on the upper beam flanges, one after the other
- in the same phase the deck elements are jointed by means of epoxy
- work with epoxy is carried out of specially trained personnel under special safety precautions

Other aspects

- there are several aesthetic programs, each designed to meet different situations
- the environmental effects are minimised and focused on continuously throughout the whole process
- the durable and sustainable materials ensure a low life cycle cost and minimise the need of maintenance and repair
- each bridge is produced for a specific lifetime over which it is maintenance-free
- after the end of the lifetime, the bridge is demolished and the materials are reused in other applications, only a small part needs to be finally disposed of

It is difficult to estimate the economic potential of the bridge concept, since it contains several features which have not been used in production before. For example, the size of the pultruded beams is larger than similar current products. On the other hand, pultrusion is one of the most cost-effective ways of producing GFRP products, while there is a need for large series to reach this low price per unit. The amount of CFRP is also significant, while CFRP is a material which is currently rather expensive. Hence, an economic estimation reveals that the initial cost today would be more than double compared to a conventional cast-in-place concrete bridge. In best case, if savings in groundwork and substructure as well as the effects of large-scale production and possible decreasing price of carbon fibres are taken into account, the bridge would cost about the same as the cast-in-place alternative. However, in a life cycle design perspective (compare e.g. Sarja (2002)) the prospective for the bridge concept looks much better due to its anticipated low life-cycle cost. In addition, the short construction time, especially on site, and the improved working conditions for labourers will add further to the benefits of the concept.

4.3.4 Conceptual design

Conceptual design stages

The conceptual design phase of the feasibility study was initiated by an estimative preliminary design carried out to outline the concept. This process ended with several possible alternatives. The four most interesting alternatives were examined in a somewhat subjective way by judging them out of about thirty different aspects. The aspects varied from economics, manufacturing, aesthetics, effectiveness, time to complete on site, erection methods and industrial potential, to purely technical matters. For each aspect, each alternative was rewarded a point which was weighted in terms of the severity of the aspect. The alternative with the highest weighted grade was chosen.

Apart from the evaluation of alternatives mentioned in the foregoing, the conceptual design phase of the feasibility study consists of the following parts. The first part concerns the identification of the critical issues and a preliminary evaluation of their consequences and possible solutions. The next part covers studies more in detail of problems that could not be solved in the previous part. The final part is FE analyses to validate the concept. These parts are presented in Paper V and summarised in the following. In addition, the laboratory experiments conducted are presented in sections 4.4 and 4.5.

One obstacle when designing FRP components is the lack of generally accepted design codes and guidelines. There are, however, some examples as summarised in e.g. Keller (2003). The code chosen is the quite comprehensive Eurocomp (1996) which, despite the fact that it is mainly valid for GFRP, is also adopted for CFRP.

A review of bridge codes can be found in e.g. Bangash (1999). The code used in the study is the Swedish Bro 2004 (2004), which is believed to cover most design situations. It is mainly the traffic loads, the load combinations and deflection requirements etc. that have been taken from the bridge code. For concrete in general, Modelcode 90 is adopted; see CEB (1993). In addition, although not used in this study, the Canadian bridge code provides for design of FRP bridge structures, see Mufti et. al (2007).

The GFRP consists of E-glass in vinyl ester matrix. The material properties for the GFRP were estimated by assuming a fibre volume fraction of 60% which is a normal level to be reached in pultrusion. All essentially bi-directional laminates were assumed, with only a small portion of the fibres (about 20%) in the diagonal directions. The CFRP is made of unidirectional IM (intermediate-modulus) PAN carbon fibres embedded in epoxy resin. The volume fraction of fibres is assumed to be 60%. The amount of steel fibres used in the CRC concrete was 6% by volume.

The most essential assumed material properties and partial safety factors used in design and FE analyses are presented in Table 9.

Table 9. Some material properties and partial safety factors used in design and FE analyses.

CFRP	**GFRP**	**Partial safety factors, FRP**	**CRC**	**Partial safety factors, CRC**
E_{xk} = 160 GPa E_{yk} = 8 GPa E_{zk} = 8 GPa σ_{xtk} = 2000 MPa σ_{xck} = -1500MPa σ_{ytk} = 45 MPa σ_{yck} = -160 MPa σ_{ztk} = 45 MPa σ_{zck} = -160 MPa	E_{xk} = 21 GPa E_{yk} = 21 GPa E_{zk} = 8 GPa σ_{xtk} = 340 MPa σ_{xck} = -160 MPa σ_{ytk} = 340 MPa σ_{yck} = -160 MPa σ_{ztk} = 8 MPa σ_{zck} = -45 MPa	<u>*ULS*</u> γ_m = 1,5 (strength) γ_m = 1,1 (stiffness) <u>*SLS*</u> γ_m = 1,3 (strength) γ_m = 1,1 (stiffness)	E_{ck} = 46 GPa f_{cck} = 125MPa f_{ctk} = 14 MPa $f_{ctk,cr}$ = 6 MPa	<u>*ULS*</u> γ_m = 1,8 (strength) γ_m = 1,44 (stiffn.) <u>*SLS*</u> γ_m = 1,0

Critical issues and suggested solutions

The conceptual design phase started with estimative calculations to capture the gross dimensions of the structural elements. Once the evaluation of the different alternatives and the subsequent choice were made, the calculations were somewhat refined before entering the finite element (FE) analyses. The preliminary design in the serviceability limit state (SLS) and the ultimate limit state (ULS) were verified with the FE analyses. Some critical design issues are summarised in Table 10.

Table 10. Summary of critical design issues.

Serviceability limit state – SLS
- deflection criteria are the issue governing the design in SLS
- crack widths for long-term loads will not influence the durability
- dynamic response was evaluated by FE analyses
- creep under sustained loading conditions and shrinkage in the concrete were considered

Ultimate limit state – ULS
- the global stress levels were considered,
- local problems were estimated and checked according to Eurocomp, e.g. stability problems

Fatigue
- fatigue are mostly related to the GFRP
- CRC and CFRP are not very sensitive to fatigue
- estimations suggest that there seem to be no fatigue problem for the structural components
- testing is proposed since the number of stress cycles for design of bridges is more than recommended for GFRP and the fatigue in the interfaces are difficult to judge

Ultimate failure
- the ultimate failure is likely to be brittle due to the elastic behaviour of the FRP
- an early warning and a somewhat semi-ductile response are expected by local loss of bond in the interface between concrete and GFRP in the vicinity of the load, followed by excessive deflections
- it could be possible to use fibre optic sensors (FOS) as an early-warning-of-failure system

Accidental load and sabotage
- the sabotage issue must be treated on a case-to-case basis, with relevant risk assessment for each location
- some geometrical design measures can be taken, e.g. to make the assessment under the bridge difficult by means of a steep slope, etc.
- a fire design based on the risk assessment must be done, ensuring that the evacuation time for people nearby is sufficient in case of an emergency
- solutions in case of fire can be physical protection of the structural components or the use of phenolic resins in the FRP
- the risk of collisions must be taken into account in the design, it must be validated that the superstructure will withstand the loss of one or several of the beams

GFRP-related issues
- GFRP exhibit creep, i.e. time-dependent deformation under constant load
- for high stress levels or elevated temperatures under long-term loads, the creep can develop into stress rupture
- a rough estimation for the bridge concept shows that the low long-term load levels (due to the low dead weight) is likely to give no problems with excessive creep, but the subject needs to be looked into more thoroughly
- it is probable that it is the creep and the physical ageing of the GFRP that determines the design life of the bridge
- stress relaxation is a phenomenon that can be present at high stress levels, which is not the case for the bridge
- GFRP in general can suffer from alkaline attacks but the vinyl ester resin combined with the E-glass should provide assurance in this respect
- stress corrosion might occur, but this is only in acid environments, which is not case here
- hygrothermal stress, i.e. internal stress as a result of variations in temperature or humidity, is counteracted by the use of symmetric and balanced laminates

Prestressing of CFRP
- possibilities to prestress the GFRP beam, by means of prestressing the CFRP profile, were roughly estimated
- a modest prestressing force would act favourably for the stresses in the beam, and it would also counteract creep effects for long-term loads
- a natural pre-camber would be achieved

Durability
- the FRP and the CRC are very durable materials
- the resistance to environmental attack such as chloride penetration, freeze thaw and carbonatisation is very good
- it has to be validated that the deck surface, especially at the joints between the elements, is absolutely watertight in order to prevent water from penetrating down to the CRC/GFRP interface with possible freeze damage as a result
- the GFRP beams might need to be drained in order to reduce the risk of humidification problems
- the reparability of FRP structures is good since they generally ensure localised damage, the repair is simply carried out on site, where the damaged parts are replaced and the structure reinforced by means of new fibres laminated onto it

Finite Element Analyses

The FE analyses conducted were done sequentially on different levels, and with different detailing according to the purpose. First, a two-dimensional analysis of one beam was carried out in order to validate the estimations done so far. Next, the scope was expanded into 3D analyses of one beam, enabling a more realistic FE modelling of the beam. Finally, in order to capture the behaviour of the whole bridge and especially the deck, a global 3D model including all beams and the deck was developed. The 3D models made use of curved shell elements, while all interfaces and joints were modelled with interface elements. Eccentric tyings, i.e. stiff connections, were used to connect the different shells and interfaces. The analyses were carried out using the general finite element program Diana; see TNO (2005). The element mesh of the global 3D model is shown in Figure 21.

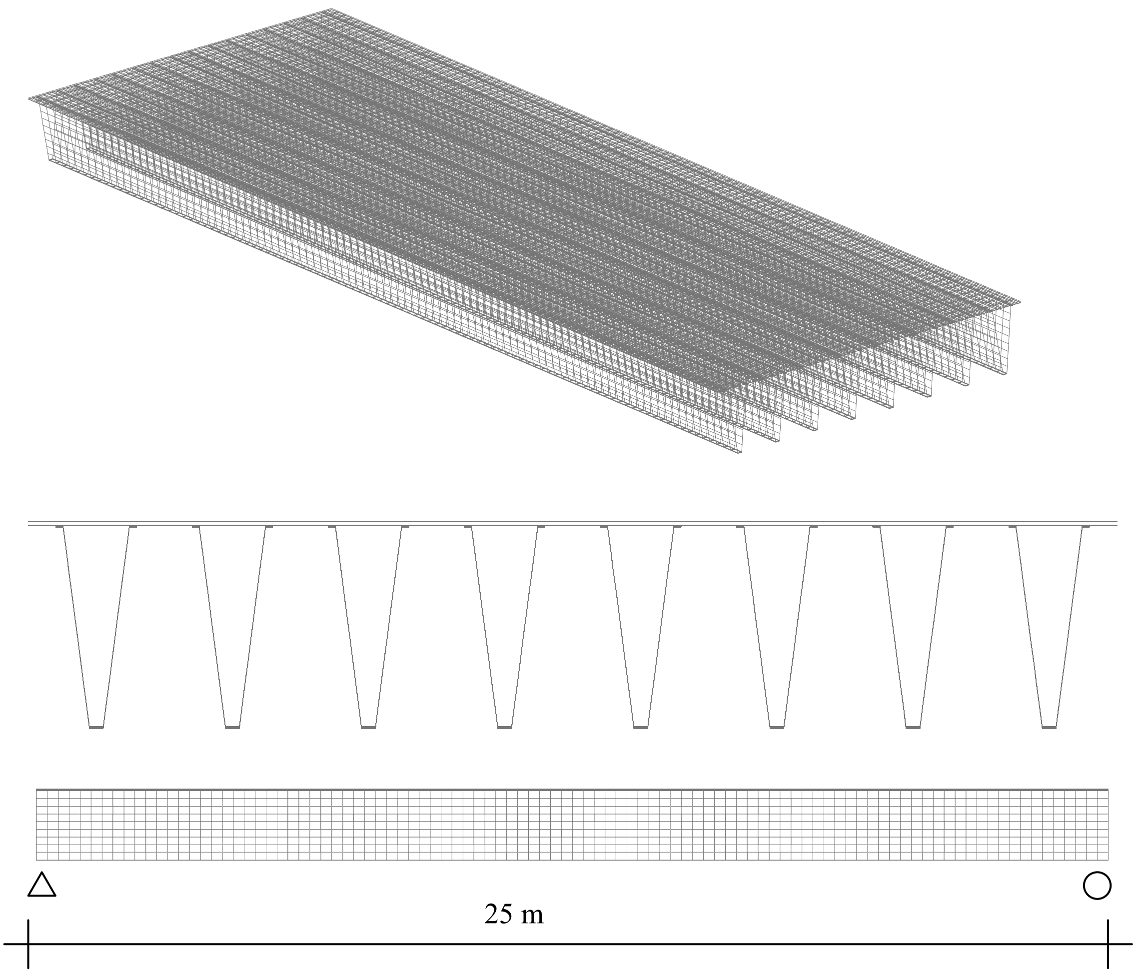


Figure 21. The element mesh of the global 3D model.

Analysis was carried out in both SLS and ULS. In addition, the modes of Eigen-frequency were evaluated and buckling analysis was performed.

The FRP components were modelled with linear elasticity in the analyses, since FRP behave linear elastic until failure.

The results from the FE analyses confirm the conceptual design and demonstrate that the bridge concept corresponds to the code requirements. Some results are presented in the contour plot in Figure 22.

The design-governing deflection issue were checked. The maximum deflection in SLS received from the analyses was 61 mm, which almost equals the deflection criterion of span-length/400 which gives 63 mm. Hence, the material properties evaluated in the conceptual design seems accurate.

The lowest mode of Eigen-frequency from the analyses was 1.4 Hz.

The issue of optimising the structure needs to be focused on in the continuation of the concept development. For example, the properties of the deck in the transversal direction must be looked into, since the current analysis indicates that its capacity is on the borderline for the worst load case in ULS.

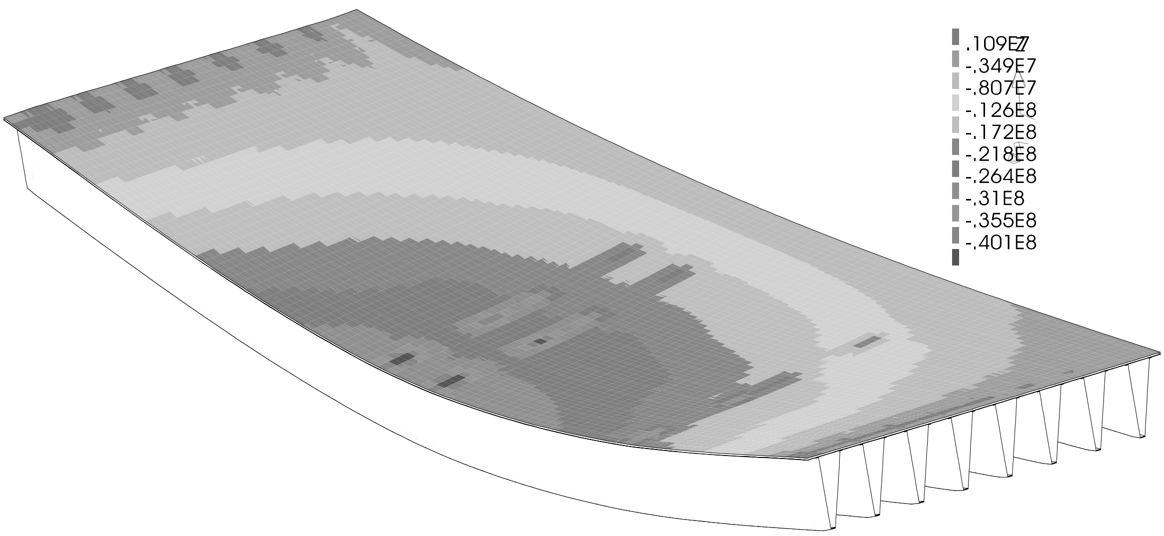


Figure 22. Contour plot of the CRC concrete in the bridge deck on the deformed shape from the FE analysis. Normal stresses [Pa] in the longitudinal (x) direction at full traffic load (two lanes with three axles each).

4.4 Experimental study of bond in bimaterial interfaces

4.4.1 General remarks

An experimental study of bond in the bimaterial interfaces of the bridge deck were carried out in order to investigate the behaviour in the joints. The summary presented here refers to Paper VI. The essential areas that were chosen for the study are the bond between concrete and GFRP in the deck and the bond in the concrete joints between the prefabricated deck elements. Especially in the first case, the bond in shear is crucial for the performance of the concept, but the bond in tension is also of interest in both cases. Hence, both shear and tension tests were conducted. The tests were carried out as direct shear and direct tension tests. To be able to simulate the joint behaviour in numerical analyses, not only the failure strength but also the deflections were measured, unlike many other tests of this kind. The experiments were aiming at creating the best bond with the simplest means. Hence, no mechanical devices (e.g. studs etc.) or special geometry (e.g. grooves, indention or ribs) were considered at this stage. A number of different suggested solutions for the two areas of interest were tested in order to be able to make suggestions about jointing technology for best performance. In addition, material parameters for the concrete were needed to verify the capacity and to model the concrete correctly in the FE analysis. Thus, materials tests on the CRC concrete were also carried out.

4.4.2 Bonded bimaterial interfaces

A short review of bond in bimaterial interfaces is presented in the following. Generally speaking there are three basic mechanisms which account for the bond in these kinds of joint interfaces between materials, namely adhesion, friction and mechanical interlocking. This is not only true for joint interfaces where concrete constitutes one or both of the adherents, but is more generally applicable when any kinds of materials are brought to cooperate through this kind of jointing.

Adhesion is a chemical mechanism which develops when two materials are brought extremely close together, close enough to develop van der Waals connections between the molecules from the different materials. The mechanism is called adsorption more specifically. The distance needed is in the range of a few Ångströms. Hence, a strong bond would be developed if two solid materials could be pressed together so closely. However, this is not possible in reality and thus one of the constituents or a layer in between needs to be initially viscous, followed by a solidification, to allow adhesion. This is the case when gluing surfaces together, e.g. with epoxy. It is also the case when fresh concrete is cast upon a solid surface. The problem however, is to maintain the bond after the viscous material has hardened. Thus, no excessive shrinkage or other effects that cause stresses strong enough to destroy the bond can be allowed during the hardening process. In addition, different materials develop bonds of different strength. The hardened material must, of course, also be of sufficient strength for the application in question.

Friction is perhaps a more understandable mechanism, and is dependent on a normal stress in the interface in order to develop. The shear friction theory has a long history and it is based on the Coulomb rule. The normal stress can be provided as an internal clamping effect or due to external forces. Most of the codes base their recommendations for shear in joints on the shear friction theory.

Mechanical interlocking is also a more comprehensible mechanism, meaning that the force is counteracted by two parts or surfaces that collide. The roughness of the substrate plays an important role in making this happen. The interlocking is of course deeply dependent on the scale, geometry and strength of the parts colliding.

There are differences between these mechanisms as regards their significance for aspects of the direction of the force transferred through the joint. For shear forces, all three mechanisms are important in different stages. For tension forces, the adhesion stands for most of the resistance contribution, since friction is usually negligible unless there is some kind of confinement (e.g. a compression parallel with the joint surface) and interlocking is usually not pronounced in this case.

Additionally, although trivial, yet another mechanism accounts for compression forces transferred through the joint, i.e. contact pressure.

In the present study, the substrate is not concrete but GFRP in the case of concrete cast joints. Hence, there are some differences compared with concrete-to-concrete joints. On the other hand, there are numerous similarities. Given the industrial environment and the proposed production for the concept in the feasibility study, there are vast possibilities to enhance the properties of the joints, i.e. creating an engineered surface.

The case of epoxy-jointed concrete surfaces is a different matter, however. Here it is the theories of adhesives that are applicable, e.g. from Eurocomp (1996).

4.4.3 Experimental programme

The choices of different test surfaces for the material interfaces between CRC and GFRP and for the epoxy-glued interfaces between two CRC concrete surfaces, are presented in Table 11. Some of the surfaces of the tested interfaces are shown in Figure 23.

Table 11. Summary of the test interfaces showing surface treatments of GFRP prior to casting and of CRC prior to epoxy-gluing.

Interfaces between CRC and GFRP	**Epoxy-glued interfaces**
Surface B: - no treatment but cleaning of the GFRP surface	**Surface A:** - as a reference test two GFRP surfaces joined by epoxy adhesive were tested
Surface C: - applying epoxy on the surface just before casting	**Surface I:** - the smooth "formwork side" was sand-blasted before the epoxy gluing
Surface D: - as above, but with an additional quartz sand coating	**Surface J:** - retardant was applied in the formwork and aggregates and steel fibre were subsequently revealed by rinsing with high water pressure prior to the epoxy gluing
Surface E: - sand coating in epoxy which was allowed to harden before casting	
Surface F: - like the preceding, but with additional specially double-bent steel fibres added to the epoxy before spreading the sand	**Surface K:** - special bent steel fibres, with one part cast into the concrete, reinforced the epoxy joint
Surface G: - special GFRP surface with non-bonded glass fibres	
Surface H: - special GFRP surface with a "peel-ply" on top; the ply was removed prior to casting, resulting in a rough sandpaper-like surface	

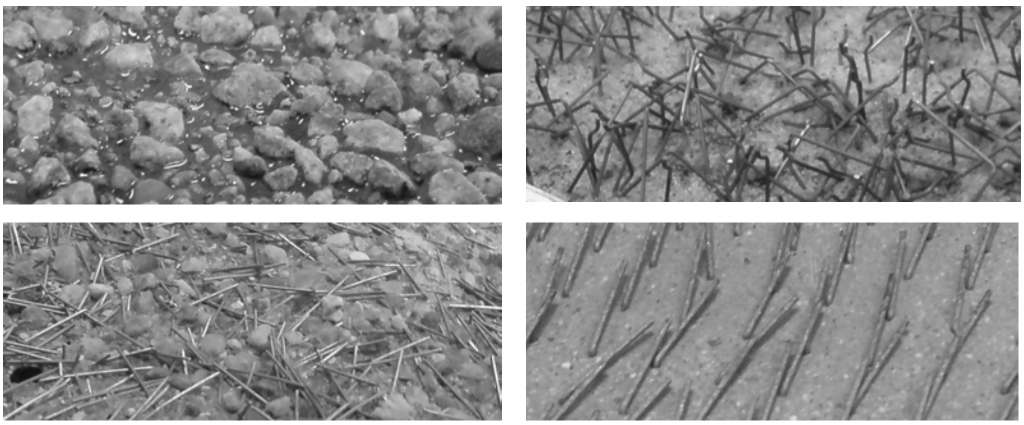


Figure 23. Some of the surfaces of the tested interfaces before casting of concrete or epoxy-gluing. Above left is surface D, above right is surface F (before removal of loose sand), below left is surface J and below right is surface K.

A test sheet was manufactured for each of the variants of surfaces mentioned in Table 11. The dimensions of the sheets were 400x400 mm^2 with a thickness of approximately 60 mm (approximately 30 mm CRC concrete or GFRP on each side of the interface). For the shear tests, almost cubic test specimens with the dimensions approximately 50x50x 60 mm^3 were sawn from the sheets with a diamond cutting edge. For the tension tests, cylindrical specimens were core-drilled from the sheets, with an inner core diameter of 50 mm down to the interface to be tested and an outer core diameter of 100 mm through the whole sheet. Test specimens before testing are shown in Figure 24.

It can be noticed that it was not possible to cut out any test specimens for surfaces B and G, since the interfaces separated due to lack of adhesion.

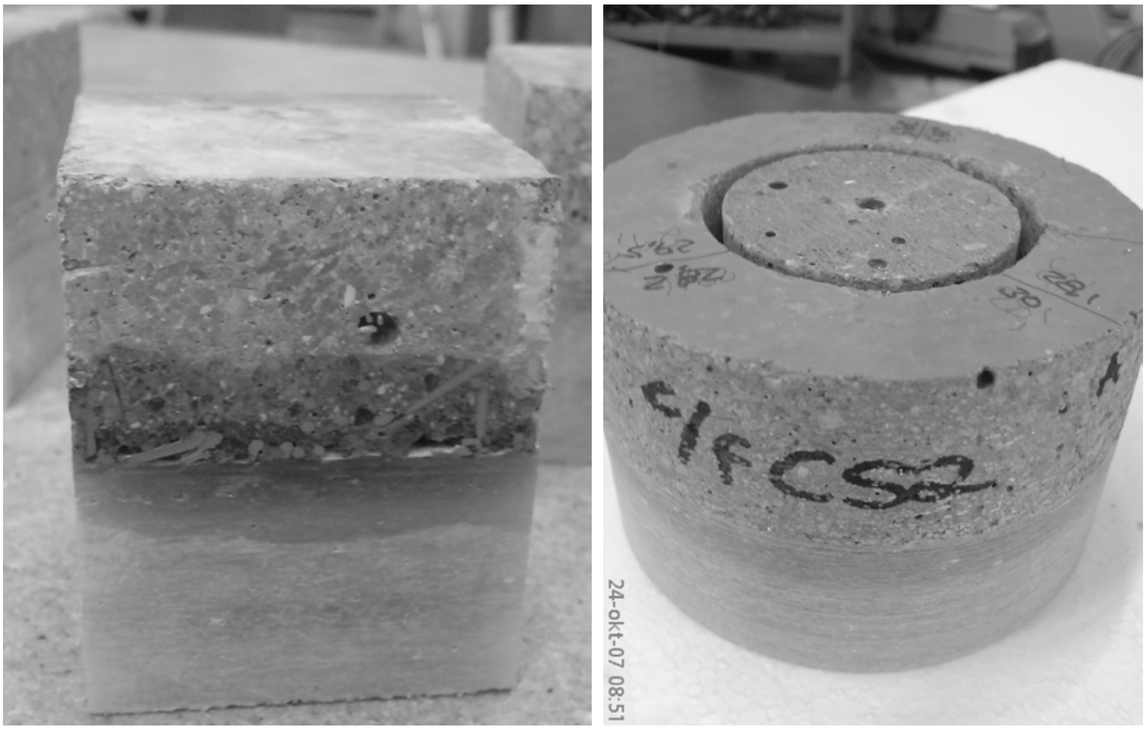


Figure 24. Test specimens before testing, to the left for shear test (test sheet F, before cutting of slots), and to the right for tension tests (test sheet C).

4.4.4 Bond tests

Performance

For shear tests, a series of at least three test specimens was sawn from the test sheet for each of the interfaces to be tested. Each specimen was mounted with epoxy glue to special steel angles in order to produce a monolithic specimen for testing. The principles for the test and the set-up is shown in Figure 25. The load and displacement were measured with a load cell and through two LVDT gauges mounted on the specimen.

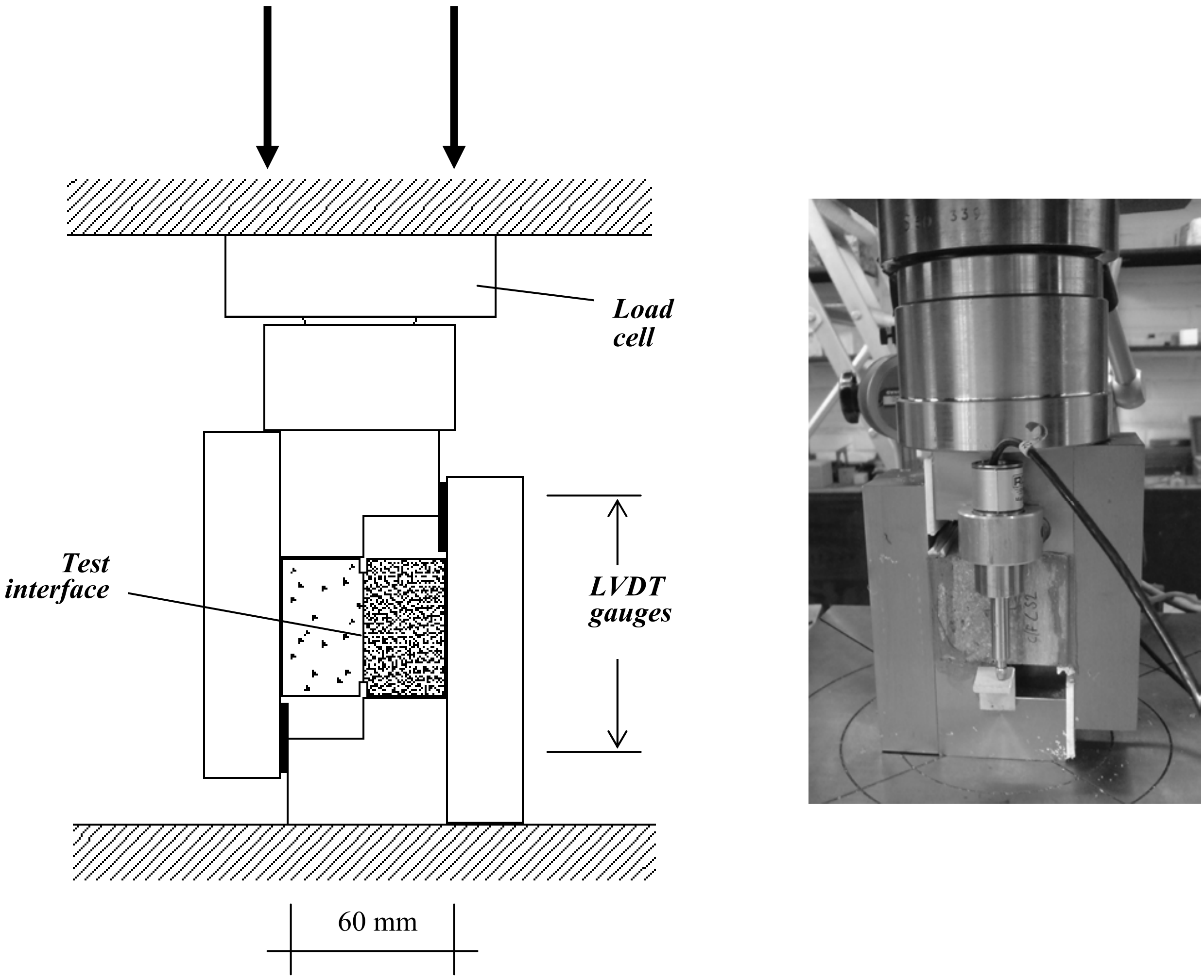
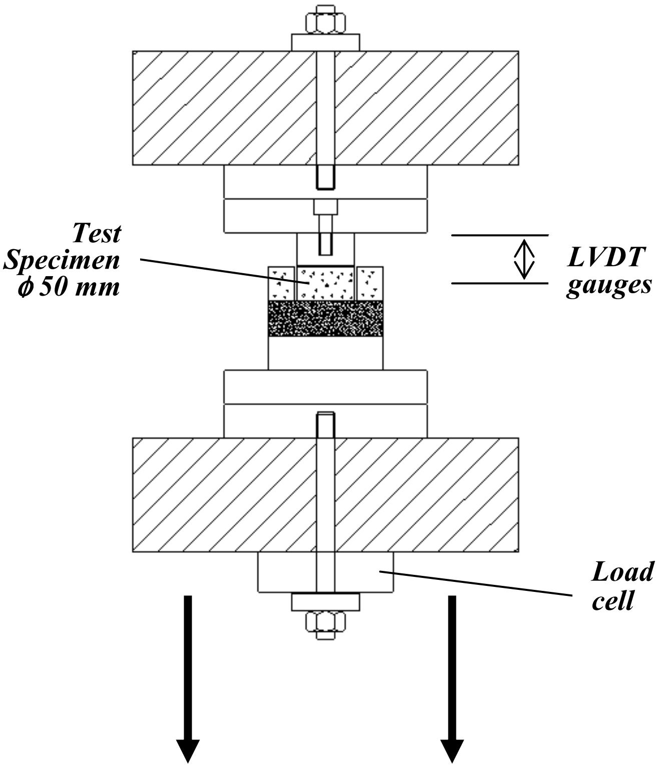


Figure 25. Test principles and set-up for shear test.

Similarly, for the tension tests, a series of at least three specimens were core-drilled from the test sheets. Each of the specimens was glued with epoxy to circular steel plates. The mounting was done in the test machine in order to ensure parallelism of the steel plates and that mounting would be done without eccentricities. The principles and the set-up for the test are presented in Figure 26. The load and displacements were measured with a load cell and through three LVDT gauges mounted on the specimen.



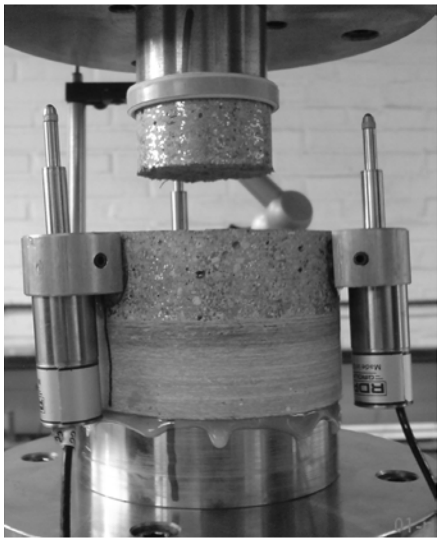


Figure 26. Test principles and the set-up for the tension tests (to the right after testing).

Results

A summary of the test results for shear and tension test are presented in Table 12, where the average failure loads and corresponding calculated average stresses for each series can be found. Diagrams for some series of the shear tests are presented in Figure 27 and for some series of the tensions tests in Figure 28. More detailed results can be found in Paper VI.

Table 12. Summary of results from the shear and the tension tests. Average failure loads and corresponding calculated average stresses for each of the series. Denotation c stands for concrete and f stands for fibre-reinforced polymer. The first capital letter defines the test series and the second stands for shear or tension tests respectively.

Shear Test Series	Average load at failure (kN)	Average calc. Shear stress (MPa)	Tension Test Series	Average load at failure (kN)	Average calc. Tension stress (MPa)
f/f AS	21,21	12,07	**f/f AT**	13,16	6,37
c/f CS	7,46	3,66	**c/f CT**	2,67	1,29
c/f DS	7,07	3,37	**c/f DT**	2,79	1,35
c/f ES	6,11	3,08	**c/f ET**	3,59	1,74
c/f FS	10,75	5,29	**c/f FT**	8,30	4,02
c/f HS	9,74	4,79	**c/f HT**	4,36	2,11
c/c IS	20,76	10,31	**c/c IT**	11,33	5,48
c/c JS	15,27	7,75	**c/c JT**	12,95	6,27
c/c KS	18,37	8,81	**c/c KT**	10,07	4,88

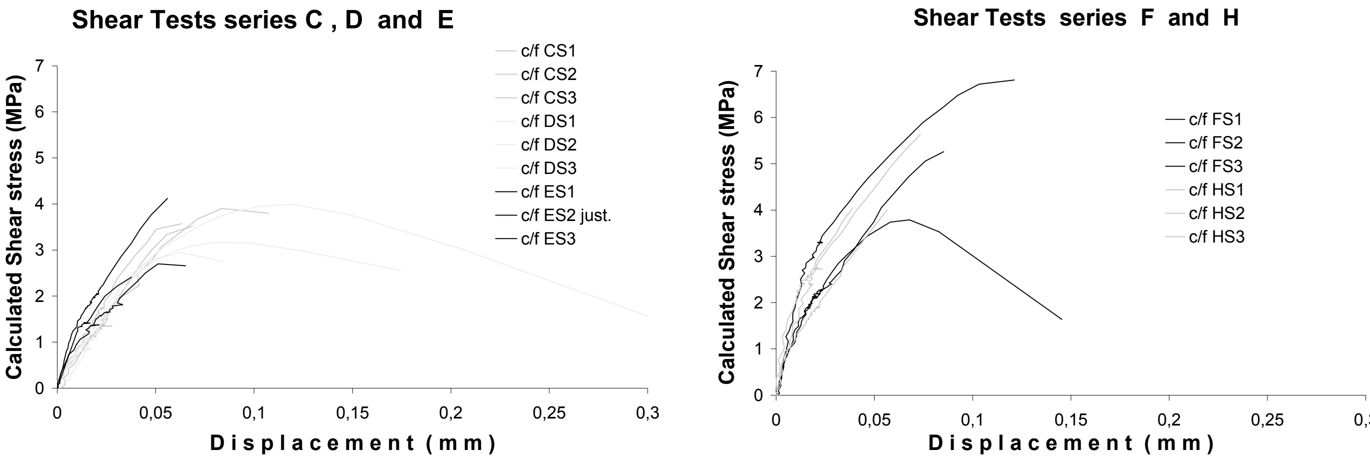


Figure 27. Test results for all interfaces between concrete and GFRP, calculated shear stress vs. displacement. To the left series C, D and E, to the right series F and H.

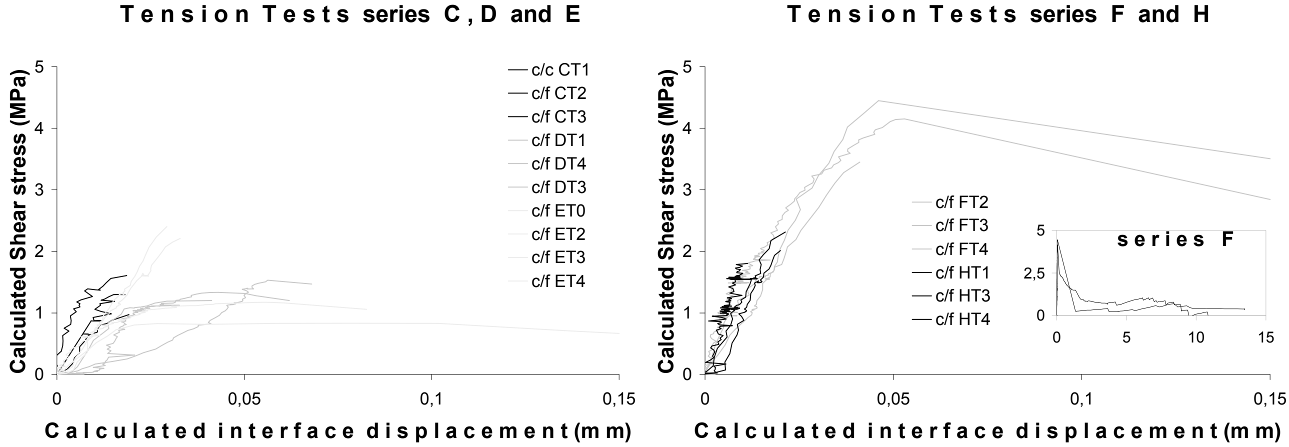


Figure 28. Test results for the interfaces between concrete and GFRP, calculated average tension stress vs. calculated average displacement in the interface. To the left series C, D and E, to the right series F and H (with cut-in descending branch for F).

4.4.5 Material tests of CRC

Tension testing

Two series of six dog-bone-shaped specimens was cast for tension testing of the CRC concrete. The first series, containing specimens DB1 – 6, were cast when performing the experimental study of bonded interfaces. The second series, containing specimens DB7 – 12, were cast in connection to the casting of the concrete in the deck of the prototype test beam. Both series are presented here.

The dimensions of the central measurement field of the dog bones were 100x50x30 (length x width x thickness). Each specimen was mounted with epoxy glue to special steel parts which were fixed in the test machine. The principles and the set-up for the test are presented in Figure 29. The load, strain and displacement were measured with a load cell and through two strain transducers and two LVDT gauges mounted on the specimen.

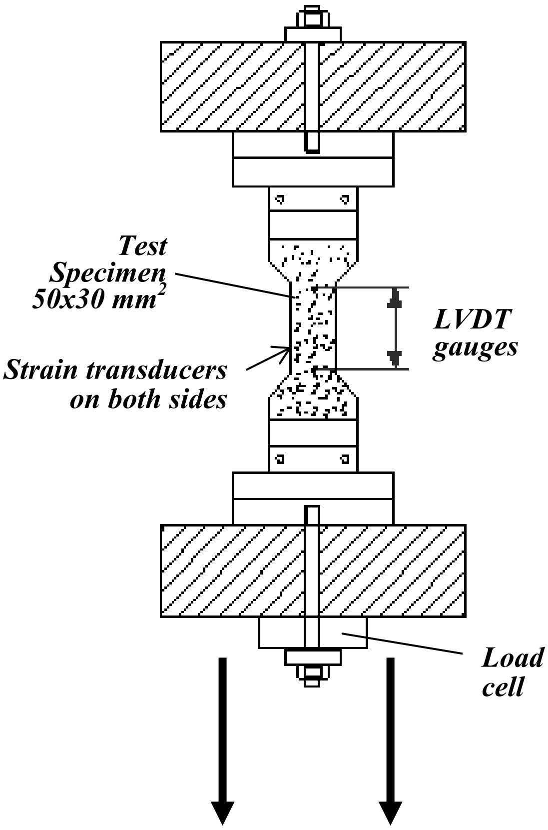


Figure 29. The principles and the set-up for the tension tests of CRC.

A summary of the test results in terms of crack load and the corresponding calculated average tension stress, along with the failure load and the corresponding average calculated tension stress, is shown in Table 13. The crack load has been estimated from the test diagrams.

Diagrams which present summaries of the test results for test specimens DB1, DB2 and DB4 are shown in Figure 30. The failure loads were lower than expected, due to uneven distribution of steel fibres in the concrete; this is further discussed in section 4.4.6.

Table 13. Summary of results from tension test of CRC concrete. DB1 – 6 from the experimental study of bonded interfaces and DB7 – 12 from the load test of the prototype beam.

Specimen	Crack load (kN)	Average Crack load (kN)	Calc. Tension stress (MPa)	Average calc. Tension stress (MPa)	Failure load (kN)	Average load at failure (kN)	Calc. Tension stress (MPa)	Average calc. Tension stress (MPa)
DB1	11,8		8,2		11,88		8,24	
DB2	10,9		7,8		11,05		7,90	
DB3	11,7	11,4	8,0	7,9	13,11	12,43	8,96	8,55
DB4	12,7		8,7		14,42		9,85	
DB5	11,3		7,7		13,56		9,25	
DB6	10,3		7,0		10,51		7,14	
DB7	-		-		-		-	
DB8	12,2		8,1		12,2		8,1	
DB9	11,7	11,5	7,4	7,4	13,7	12,0	8,7	7,7
DB10	9,7		6,2		10,0		6,4	
DB11	12,2		7,8		12,2		7,8	
DB12	11,9		7,6		11,9		7,6	

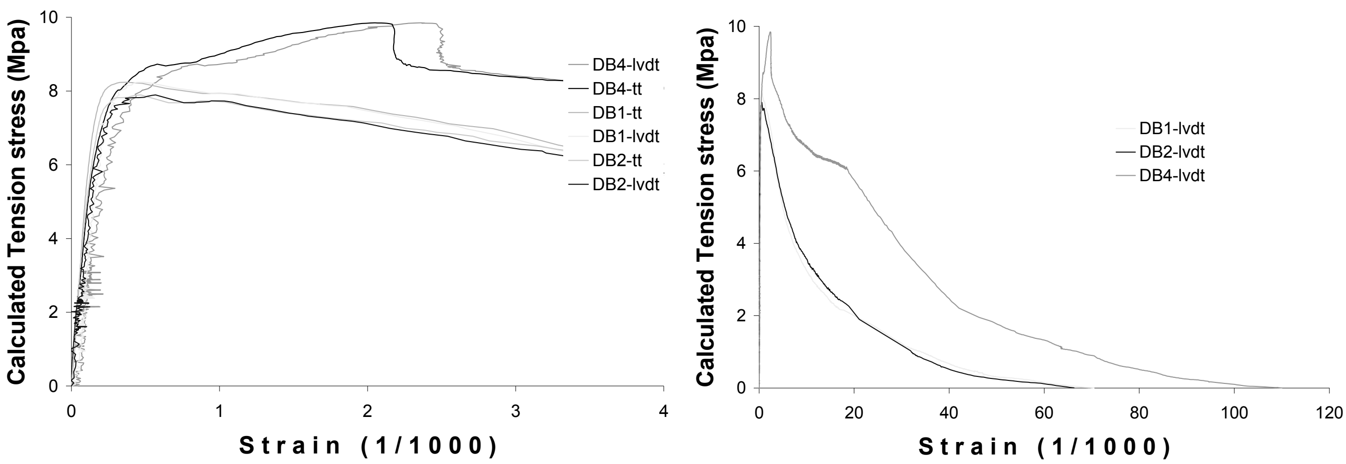


Figure 30. Test results for specimens DB1, DB2 and DB4, calculated tension stress vs. strain. To the left average measurements for strain gauges and LVDT gauges (calculated strain), to the right the descending branch for the LVDT gauges. Notice the different scales.

Compression tests

Similar to the tension tests, compression tests were also conducted both in the experimental study of bonded interfaces and in the load test of the prototype beam. Hence, two times three series of three cylinders each were cast for compression tests and evaluation of elasticity modulus (on two series). The dimensions of the cylinders were ϕ100x200 mm^2. A summary of the results are presented in Table 14.

Table 14. Summary of results from the compression tests of CRC concrete. Series A1 – A3 are from the experimental study of bonded interfaces and series B1 – B3 are from the load test of the prototype beam.

Cylinder	Stress at failure f_{cc} (MPa)	Average f_{cc} (MPa)	Modulus of Elasticity E_c (GPa)	Average E_c (GPa)	Age (days)
A1:1 **A1:2** **A1:3**	145,0 147,2 151,6	147,9			29
A2:1 **A2:2** **A2:3**	157,7 153,5 161,0	157,4	52,16 55,50 56,80	54,8	56
A3:1 **A3:2** **A3:3**	162,2 165,1 133,9	153,7			63
B1:1 **B1:2** **B1:3**	153,4 153,1 151,6	152,7	58,0 58,4 54,4	56,9	50
B2:1 **B2:2** **B2:3**	- 155,9 157,2	156,5			50
B3:1 **B3:2** **B3:3**	155,6 157,3 157,2	156,7			50

4.4.6 Discussion and conclusions

There are difficulties encountered with regard to keeping an even load over the whole test surface in the kind of bond tests that has been performed. That is primarily due to the composition of the test surfaces, where most of the surfaces, by their nature, encompass parts that cause stress concentrations. But it is also due to the small dimensions of the test specimens, where stress concentrations will have a significant effect on the result. A third factor is the problem of designing the tests so as to keep an even distribution of loads throughout the tests. Most sensitive to the aforementioned are the tension tests, but there are also considerable effects in the shear tests. It is easy to apprehend these circumstances by evaluating the difference in the measured displacement between the different LVDT gauges in each test. There are large differences in the displacements for most of the tension tests and for about 50% of the shear tests. Not surprisingly, there has been an obvious trend that tests with uneven displacements show a lower failure load than those with even displacements. Consequently, this affects the scatter in the test, which is rather large in some series.

The failure loads were lower than expected for both series of tension tests of CRC, while the specimens did not show the strain hardening response that was supposed. The reason for this is probably the uneven distribution of steel fibres which was observed in the failure surfaces, which seems to be the result of too much vibration when casting the dog bone specimens, causing separation of fibres. As a consequence there was not an adequate amount of steel fibres present in the failure surfaces. An evidence for this can be observed when calculating the fibre efficiency factor for the concrete $\boldsymbol{n_b}$; see e.g. Löfgren (2005). The factor is calculated according to the following.

$$n_b = \frac{N_b \cdot A_f}{A \cdot V_f}$$

where n_b = the fibre efficiency factor
N_b= the amount of fibres bridging the failure surface
V_f= volume fraction of fibres
A_f= cross-sectional area for a fibre
A = area of the failure surface

This factor is 1.0 for one-dimensional distribution, and 0.64 and 0.5 for two- and three-dimensional distribution respectively. The geometry of the dog bone specimens should account for a distribution somewhere in between two- and three-dimensional. For specimen DB1 the amount of fibres in the failure surface was calculated to be 229, which gives

$$n_b = 0{,}32$$

Although the scatter can be quite significant, this clearly indicates that there is not a sufficient amount of fibres present in the failure surface.

Regarding the results from the shear and tension tests, the following comments can be made.

The best performance for the concrete over-layered surfaces was achieved for test sheet c/f F (compare Table 11), as expected, both in shear and in tension. It could be interesting to study how much the amount of the double-bent fibres could be decreased while still arresting a failure surface in the epoxy. In the current tests, the addition of bent fibres was 12500 per m^2, which theoretically corresponds to an upper limit for the tension failure stress of 7.8 MPa (calculated as the failure load for the fibres per area unit).

The test series from sheet c/f H (compare Table 11), showed better performance than all other series apart from test sheet F, which is especially interesting in view of the simple arrangements. An optimisation of the "peel-ply" type could perhaps enhance the performance even more. The test series for this sheet also showed a reasonable low scatter in the results.

For the epoxy-jointed surfaces, the test results are relatively equal for the three test sheets. Sheet c/c I (compare Table 11), demonstrates even results with a low scatter for both shear and tension tests.

The performance for sheet c/c K (compare Table 11) could have been enhanced by increasing the amount of cast-in fibres or choosing different fibres, but this is hardly interesting in view of the good performance of sheet I. The method would perhaps better suit the situation with concrete-to-concrete joints, the fibres being straight in that case and the protruding part being cast in.

The most suitable surface treatment for the prototype test beam seems to be surface H (special GFRP surface with a "peel-ply" to be removed prior to casting), to judge from the tests. It possesses a sufficient capacity and has a low distortion among its test results. This was also the surface chosen for in the load test, compare next section.

4.5 Assessment of a prototype test beam

4.5.1 General remarks

The aim of the laboratory load test and FE analyses presented in this section is to assess a prototype bridge beam of the *i-bridge* concept. A more thorough description can be found in Paper VII. The main focus of the structural test is to demonstrate the structural performance with special attention to the composite action in the deck, to follow and infer the failure mode, and to validate the correctness of the numerical tools used.

Of course, since this is a unique test the results cannot be statistically verified. Additionally, the test does not tell us anything about the behaviour in the transverse direction, i.e. how the deck performs in that direction. Moreover, since the test beam is scaled down relative to the real structure, there might be a discrepancy in behaviour between the two. However, this is checked to some extent by comparing the numerical analyses of the two structures.

4.5.2 The prototype test beam

The beam prototype was composed similarly to the beams in the *i-bridge* concept (compare section 4.3); only its dimensions had been scaled down in some aspects. The span of the test beam was 5.0 m, whereas the span in the concept is 25 m. The total height was lowered from 1580 mm to 820 mm and the thickness of the CRC concrete was decreased from 70 mm to 20 mm. The width of the deck and the distance between the webs were decreased correspondingly, while it was decided to keep the thicknesses of the GFRP and the capacity of the CFRP intact as in the concept. This was mainly because an aim of the test was to validate the composite action between concrete and GFRP. Hence the FRP parts would not be critical, and testing the phenomenon of composite action could more easily be achieved in the test. In addition, the tolerances of the FRP components justified a certain margin. But a further practical reason was to be able to handle the structure and to perform the test with a reasonable magnitude of load. A cross-section and a picture of the prototype beam are shown in Figure 31. The beam design and manufacturing are summarised in Table 15. The beam was load-tested 49 days after casting.

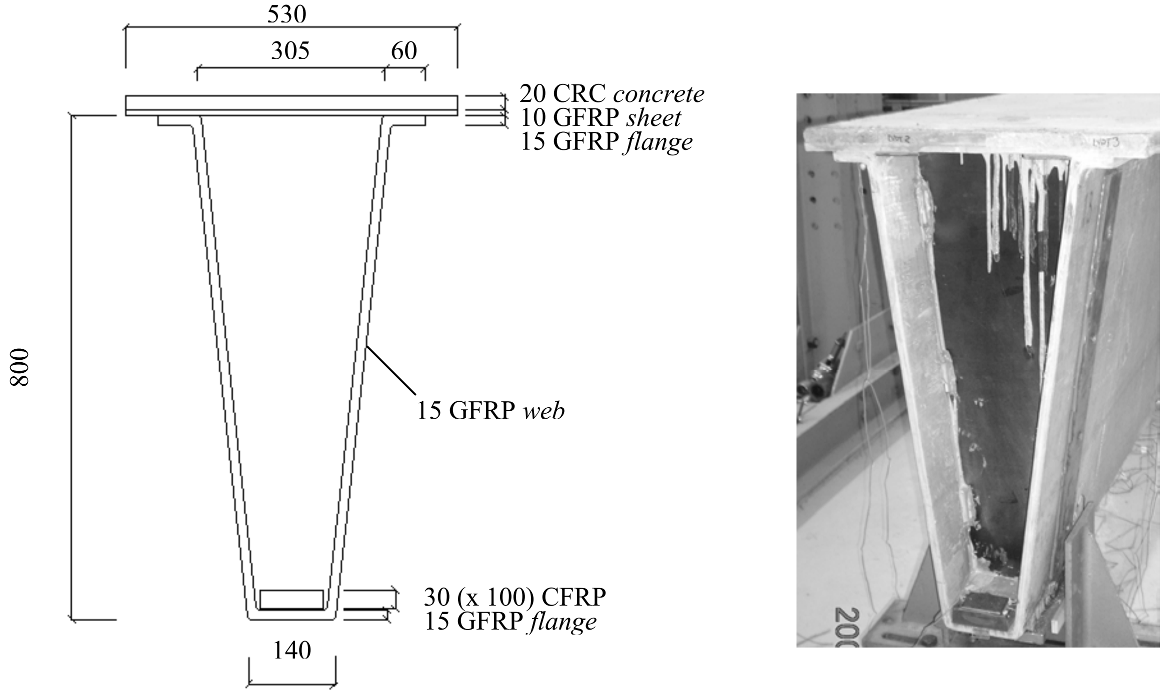


Figure 31. The cross-section of the prototype beam (to the left) and a picture showing the beam after casting of the concrete in the deck (to the right). Dimensions in mm.

Table 15. Summary of the design and manufacturing of the prototype beam.

General - the FRP parts were joined by means of epoxy adhesive - the amount of steel fibres used in the CRC concrete was 6% by volume - the surface of the GFRP deck sheet were performed according to surface H from the study of bonded interfaces - the FRP beam was manufactured in a shop - the CRC concrete was cast in the laboratory - crossbeams were cast at beam-ends above supports when casting the deck - for the reason of load transfer between the crossbeam and the GFRP web, double steel plates with welded steel studs on the inner plate – to be cast into the crossbeams – were bolted and glued by means of epoxy adhesive to the webs - bearing plates of steel were epoxy-glued to the bottom flange at supports	***The CFRP*** - was made from prepreg of unidirectional high strength PAN carbon fibres embedded in epoxy resin - the volume fraction of carbon fibres was 60% ***The GFRP*** - consisted of E-glass in polyester matrix - material properties for the GFRP were evaluated by assuming a fibre volume fraction of 50% - essentially bi-directional laminates with only a small portion of the fibres (about 20%) in the diagonal directions - the volume fraction of fibres was differentiated across the thickness of the laminate, so that the amount of longitudinal fibres was increased in the middle half while the amount of transverse fibres was increased correspondingly in the two outer fourths close to the skin - the GFRP components were made by hand lay-up and vacuum bagging using polyester resin

4.5.3 Finite element analysis

Numerical analyses were conducted to simulate both the test beam and a full-size beam spanning 25 m. This was done in order to ensure that the test beam behaves correspondingly when loaded to failure, and that the phenomena observed in the test also are those that can be expected in the full-scale structure.

The analysis was carried out with the general finite element program Diana; see TNO (2005). Both FE models were done in 3D using curved shell elements, while all material interfaces and epoxy joints were modelled with interface elements. The different shells and interfaces of the models were connected with eccentric tyings, i.e. stiff connections. The FRP were modelled with the Hoffman failure criteria, while the concrete was modelled based on smeared cracking and total strain, and the plasticity model of Thorenfeldt accounted for the non-linearity of concrete in compression. In some analyses the concrete were modelled with isotropic plasticity and von Mises failure criteria. Additionally, the interface elements were modelled with multi-linear relationships; compare TNO (2005).

Since no materials testing could be conducted for the FRP parts, there is a certain level of uncertainty for the calculated material properties. However, due to the chosen scaling of the beam, this is not believed to be critical except for the shear resistance in the GFRP webs. The assumed material parameters used in the FE simulations are presented in Table 16. The element mesh of the test beam model is shown in Figure 32.

Table 16. Material properties used in the analysis.

CFRP	**GFRP**	**CRC**	**Epoxy (interface)**	**CRC - GFRP interface**
E_x = 135 GPa E_y = 10 GPa E_z = 10 GPa G_{xy} = 5,0 GPa G_{yz} = 3,0 GPa G_{xz} = 5,0 GPa ν_{xy} = 0,30 ν_{yz} = 0,52 ν_{xz} = 0,30 σ_{xt} = 2200 MPa σ_{xc} = -1500 MPa σ_{yt} = 54 MPa σ_{yc} = -186 MPa σ_{zt} = 54 MPa σ_{zc} = -186 MPa τ_{xy} = 85 MPa τ_{yz} = 90 MPa τ_{xz} = 120 MPa	E_x = 21,1 GPa E_y = 21,1 GPa E_z = 10,7 GPa G_{xy} = 5,1 GPa G_{yz} = 5,1 GPa G_{xz} = 5,3 GPa ν_{xy} = 0,22 ν_{yz} = 0,22 ν_{xz} = 0,21 σ_{xt} = 340 MPa σ_{xc} = -163 MPa σ_{yt} = 340 MPa σ_{yc} = -163 MPa σ_{zt} = 76 MPa σ_{zc} = -76 MPa τ_{xy} = 25 MPa τ_{yz} = 25 MPa τ_{xz} = 25 MPa	E_c = 54,8 GPa ν = 0,24 f_{cc} = 150MPa f_{ct} = 14 MPa $f_{ct,cr}$ = 7 MPa g_f = 18 kN/m	<u>*normal direction*</u> D_{11}= 53 GPa/m σ_{nt} = 8 MPa δ_{nt} = 0,15 mm σ_{nc} = -60 MPa δ_{nc}= 1,125 mm <u>*shear direction*</u> D_{22}= 53 GPa/m τ_{xy} = +/- 12 MPa δ_{xt} = +/- 0,225mm	<u>*normal direction*</u> D_{11}= 100 GPa/m σ_{nt} = 2 MPa δ_{nt} = 0,02 mm σ_{nc} = -70 MPa δ_{nc}= 0,065 mm <u>*shear direction*</u> D_{22}= 89 GPa/m τ_{xy} = +/- 5 MPa δ_{xt} = +/- 0,056mm

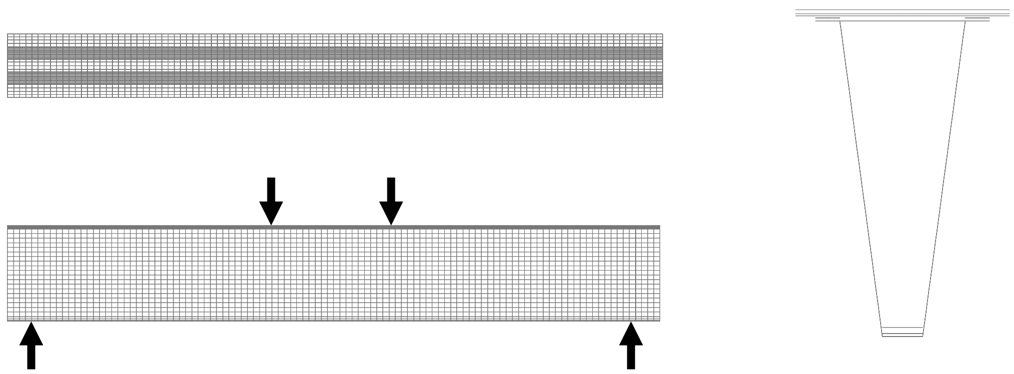


Figure 32. The mesh of the 3D FE model of the prototype beam, from above, a side view and the cross section.

Non-linear FE analyses were performed for both models. Comparison of the results from the analyses of the two models reveals a reasonable conformity of behaviour between the models, especially concerning the composite parts of concrete and GFRP. The envisaged failure mode for the full-size beam was a local bond failure in shear within the bimaterial deck interface in the vicinity of the load, while for the test beam the expected failure was a combination of compression failure in the CRC concrete and a local bond failure close to the loading point. There was a possibility that the shear stresses in the webs also could become critical due to the decreased cross-sectional area of the webs, resulting from the down-scaling of the beam and the uncertainties in the material properties. However, there should be no risk of shear failure in the web if the adopted material properties were of the right magnitude.

Contour plots from the two beams' models where concrete were modelled with isotropic plasticity are presented in Figure 33 and Figure 34.

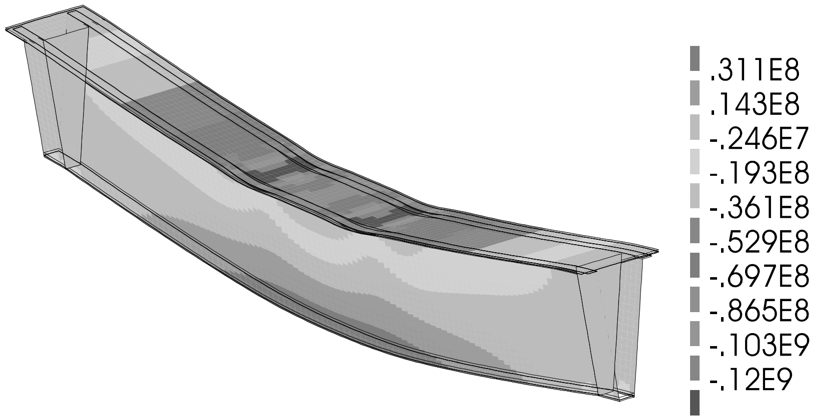


Figure 33. Contour plot of the deformed shape from FE analysis of the 5 m span prototype beam. Normal stresses [Pa] in longitudinal (x) direction at a load of 2 x 430 kN (placement of load according to Figure 35).

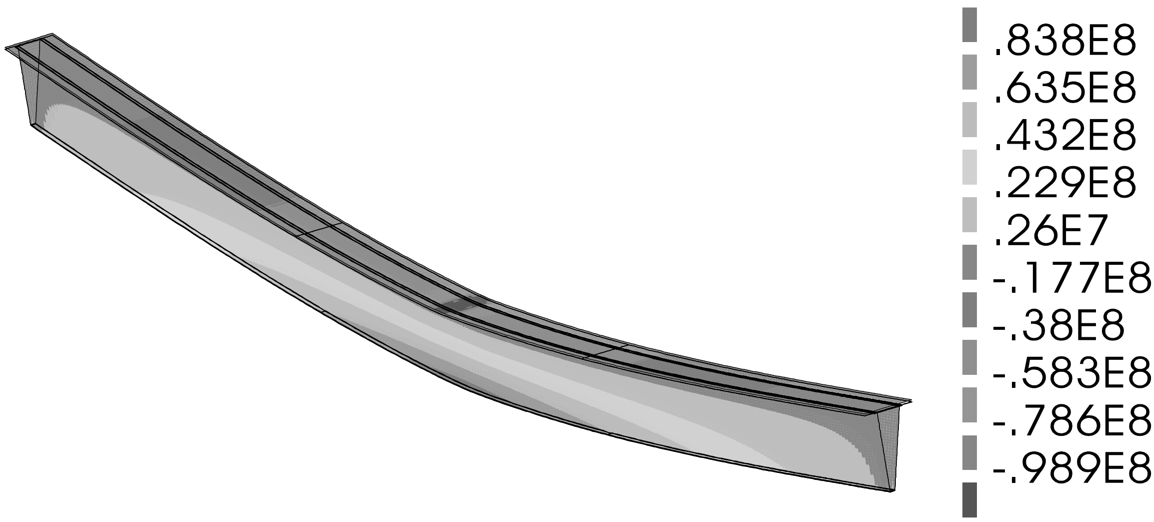


Figure 34. Contour plot of the deformed shape from FE analysis of the 25 m span full-scale bridge beam. Normal stresses [Pa] in longitudinal (x) direction at a central load of 900 kN.

Additionally, an analysis for control of Eigen values for buckling was conducted, stating that there would be no risk of buckling during the test. This was further verified by calculations according to Eurocomp (1996). Furthermore, from the FE analyses it was concluded that shrinkage would not constitute a problem since the maximum stress from shrinkage would be about 1 MPa. In addition, the shear stress from shrinkage in the bimaterial interface in the deck would act favourably, i.e. in the opposite direction, compared to the shear stresses from loading.

4.5.4 Laboratory test

Test performance

The beam was loaded in four-point bending. The set-up can be seen in Figure 35 and Figure 36. Strain transducers were mounted on both sides of the interface between the GFRP plate and the CRC, i.e. on the surface of the GFRP and facing down on short reinforcing bars cast into the CRC concrete. In addition, strain was also measured by means of strain transducers on the surface of the FRP and the CRC. Deflections and displacements were measured with LVDT gauges. Load cells accounted for the load measurement.

In addition, material tests of the CRC concrete were done. These have been presented in section 4.4.5.

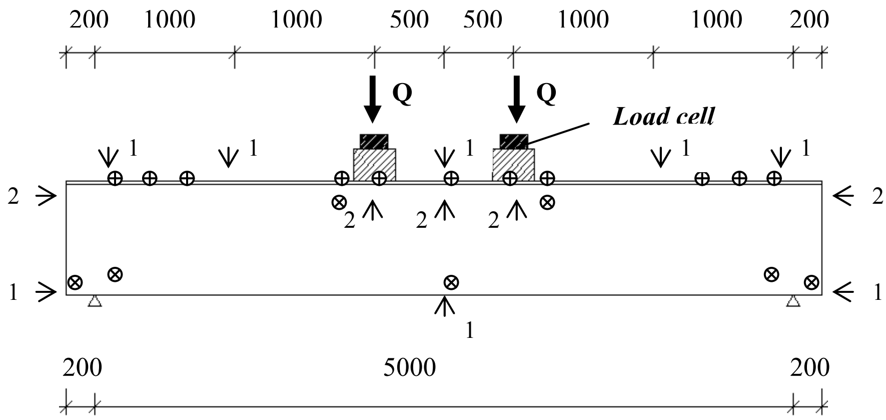


Figure 35. Set-up for the load test. Arrows denote LVDT transducers (number stands for one transducer in centre of beam and two transducers on corresponding sides respectively). Cross denote schematic placement of strain gauges (vertical cross for gauges in the deck and diagonal cross for gauges on the FRP beam).

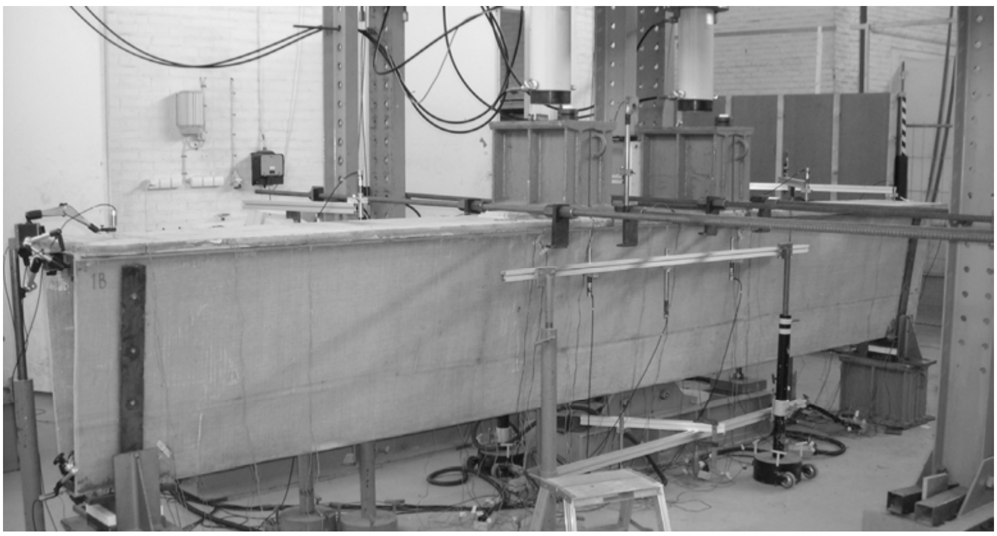


Figure 36. The prototype beam before loading.

Test results

The load test showed satisfactory agreement with the FE analyses. Thus, the structural performance predicted from the analyses could be verified. A comparison of the test results and the FE analyses is made in section 4.5.5. Some of the results from the test are presented in Figure 37 and Figures 39–41. More detailed results can be found in Paper VII. The presented results in the diagrams do not include effects of dead weight, etc., since the measurements are set to zero at the beginning of the loading in the evaluation.

The failure load reached was 2 x 429 kN and the failure was due to delamination in the GFRP plate in the deck, which was an unexpected failure mode. The failure was unfortunately induced by a transportation damage causing delamination of the plate, which was repaired prior to testing the beam. The defect can be noticed from Figure 37, showing the differential displacement between the bottom of the GFRP deck sheet and the centre of the CRC overlay at support 2. A significant displacement takes place on side A of the beam, which was the side that was damaged in the proximity of support 2 during transportation, while the displacement is practically zero on side B. In addition, no noteworthy differential displacements were measured at support 1. The beam after loading to failure is shown in Figure 38.

However, the expected failure, a combination of compression failure in the CRC and local bond failure in the bimaterial interface at the loading points in the deck, was just about to be reached when the beam failed, and the beam behaved in accordance with what was foreseen during the loading. For example, calculating the compression stress in the top of the CRC concrete at mid-span from the measured strain at failure (compare Figure 39), one obtains about 120 MPa in compression, hence demonstrating the vicinity of the ultimate compression stress. But in some respects, such as uneven distribution of strain and stresses over the cross-section, the damage is likely to have influenced the results, although it is not possible to judge how large this influence was. Nevertheless, in all essentials, apart from not reaching the expected failure mode, it is believed that the load test was successful and that the objectives of the test were achieved.

Subsequent the failure the load dropped instantaneously to 2 x 258 kN (compare e.g. Figure 40), while the load was carried solely by the FRP beam without composite contribution from the deck. The thin part of the GFRP deck sheet, still connected to the beam (below the delamination), exhibited severe compression buckling.

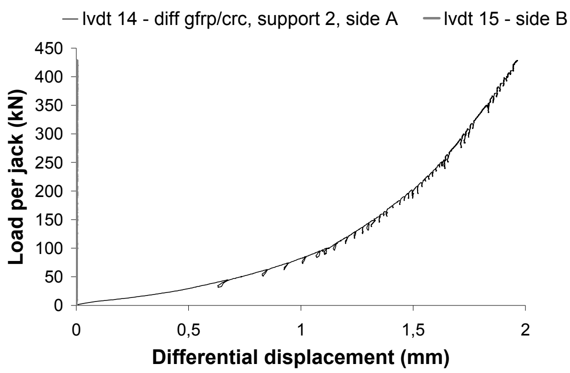


Figure 37. Differential displacement between the bottom of the GFRP deck sheet and the centre of the CRC overlay at support 2. Notice the significant displacement taking place on side A of the beam (the side which was damaged during transportation), while the displacement is practically zero on side B (the graph is almost parallel to the load axle).

Figure 38. The test beam after loading to failure.

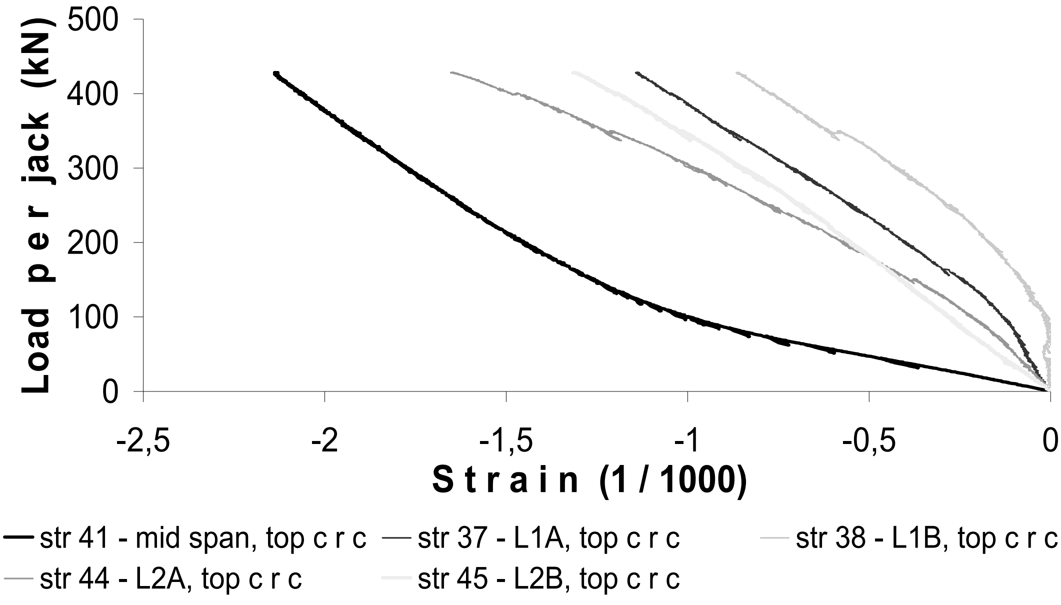


Figure 39. Strain measurements at mid-span and at the loading points for the top surface of the CRC. The mid-span strain was measured in the centre of the beam, while strains at the loading points were measured above the centre of the beam flanges on both sides just outside the load distribution device (towards the supports).

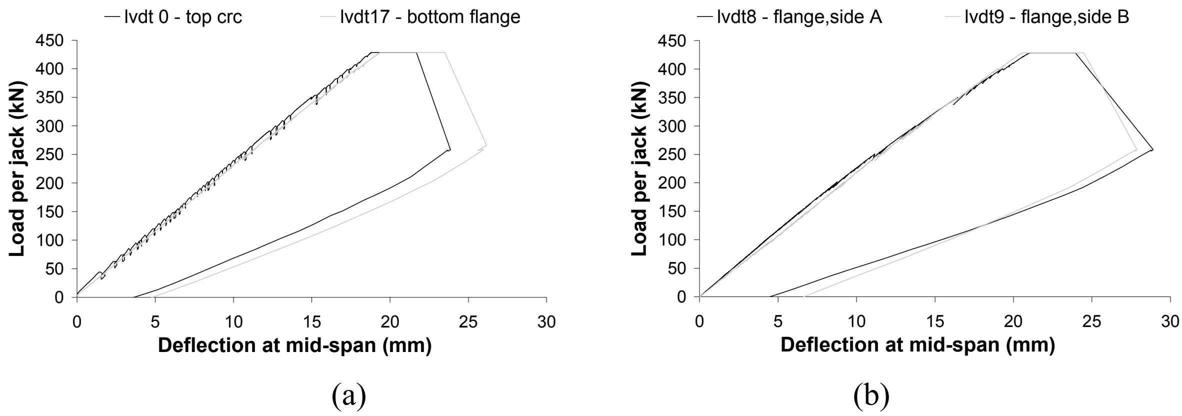


Figure 40. Load–deflection curves at mid-span; (a) for the top and bottom in the centre of the beam, and (b) for the underside of the GFRP deck sheet at the edges.

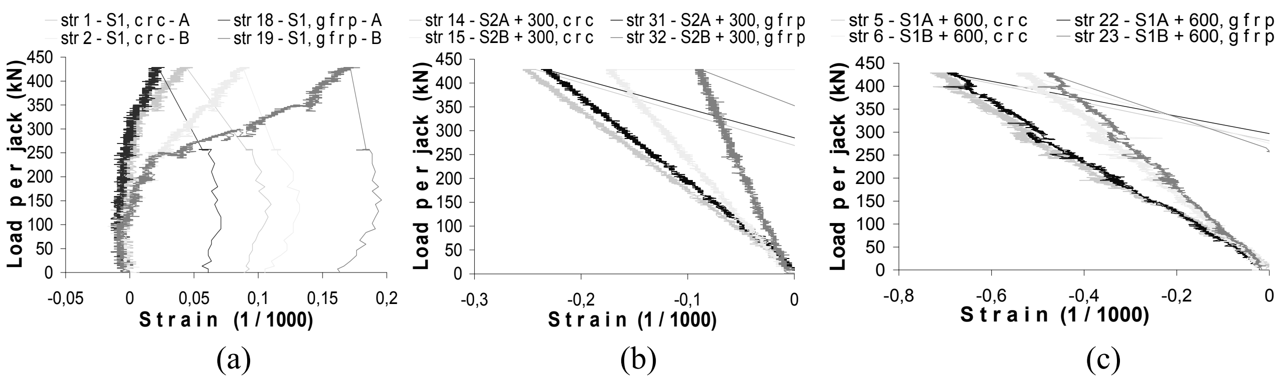


Figure 41. Strain measurements at the corresponding sides of the bimaterial interface in the deck for side A and side B of the beam (measured above the centre of the beam flanges) at various distances from the supports: (a) at support 1, (b) 300 mm from support 2 and (c) 600 mm from support 1.

4.5.5 Comparison of results and discussion

As has been mentioned, the load test exhibited reasonable agreement with the FE analyses. Thus, the structural performance predicted from the analyses was verified as can be concluded from the comparison of the test results and the FE analyses with isotropic plasticity of concrete, shown in Table 17 and Table 18.

Table 17. Comparison of deflections at failure load from the load test and the FE analyses. The failure load was 2 x 429 kN and the load in the FE analyses was 2 x 430 kN.

Placement	Load Test	FE Analyses	Placement	Load Test	FE Analyses
	Deflections (mm)	**Deflections (mm)**		**Deflections (mm)**	**Deflections (mm)**
top-face CRC, centre of beam			***bottom-face GFRP-sheet***		
mid-span (0)	18,8	18,7	mid-span-A (8)	21,1	18,8
1/5-point-S1 (5)	10,5	10,4	mid-span-B (9)	20,6	18,8
1/5-point-S2 (12)	10,6	10,4	L1A (6)	20,3	18,9
			L1B (7)	20,2	18,9
bottom of beam			L2A (10)	19,7	18,9
mid-sp.-gfrp (17)	20,2	18,6	L2B (11)	20,3	18,9

Notations: S – support, L – loading point, A –side A of beam, B – side B of beam, ( ) – number of LVDT gauges. The presented deflections are compensated for by settlements in the support.

As seen in the tables, the scatter between the test and the analyses is sufficiently low compared to the uncertainties in the material parameters, especially for the GFRP, as has been mentioned. Hence, it can be stated that the adopted parameters are in the realistic range, e.g. the real moduli of elasticity should not differ much from those used in the FE analyses.

Table 18. Comparison of stresses and strains at failure load in the global longitudinal direction from the load test and the FE analyses. Load test stresses are calculated from the strains using the average measured elastic modulus for CRC and the elastic modulus assumed in the FE analyses for FRP. The failure load was 2 x 429 kN and the load in the FE analyses was 2 x 430 kN.

Interface in the deck	**Load Test**		**FE Analyses**		**Material faces**	**Load Test**		**FE Analyses**	
	Strain ε_x (‰)	**Stress** σ_x **(MPa)**	**Strain** ε_x (‰)	**Stress** σ_x **(MPa)**		**Strain** ε_x (‰)	**Stress** σ_x **(MPa)**	**Strain** ε_x (‰)	**Stress** σ_x **(MPa)**
300 mm from					***top-face CRC***				
support, CRC					L1A-crc (37)	-1,432	-81,4		
S1A-crc (3)	-0,220	-12,5			L1B-crc (38)	-0,868	-49,4	1,79	-98,7
S1B-crc (4)	-0,329	-18,7	-0,233	-12,8	L2A-crc (44)	-1,650	-93,3		
S2A-crc (14)	-0,254	-14,5			L2B-crc (45)	-1,322	-75,2		
S2B-crc (15)	-0,174	-9,9			mid-sp.-crc (41)	-2,134	-121,4	-1,31	-72,4
300 mm from					***bottom-face***				
support, GFRP					***GFRP-sheet***				
S1A-gfrp (20)	-0,232	-4,9			L1A-gfrp (48)	-1,833	-38,7		
S1B-gfrp (21)	-0,264	-5,6	-0,231	-4,9	L1B-gfrp (49)	-0,828	-17,5	-0,976	-19,2
S2A-gfrp (31)	-0,235	-5,0			L2A-gfrp (52)	-1,165	-24,6		
S2B-gfrp (32)	-0,090	-1,9			L2A-gfrp (53)	-0,838	-17,7		
at load, CRC					***bottom-face***				
L1A-crc (7)	-0.056	-3.2			***GFRP-flange***				
L1B-crc (8)	-1,541	-87,7	-1,08	-58,3	mid-span-A (50)	-1,541	-32,5	-1,45	-30,2
L2A-crc (10)	-1,515	-86,2			mid-span-B (51)	-1,397	-29,5		
L2B-crc (11)	-1,088	-61,9							
mid-span-crc(9)	-1,815	-103,3	-1,45	-79,7					
at load, GFRP					***bottom of beam***				
L1A-gfrp (24)	-1,363	-28,8			mid-sp.-gfrp (71)	2,274	48,0	2,22	47,2
L1B-gfrp (25)	-1,823	-38,5	-1,29	-25,7	S1-top-cfrp (70)	0,134	18,1	0,44	59,7
L2A-gfrp (27)	-1,797	-37,9			S2-top-cfrp (72)	0,320	43,2	0,44	59,7
L2B-gfrp (28)	-1,302	-27,5							
mid-sp.-gfrp(26)	-1,593	-33,6	-1,46	-30,7					

Notations: S – support, L – loading point, A – side A of beam, B – side B of beam, ( ) – number of strain transducers.

In all essentials, the prototype beam behaved linear elastic till failure, as expected. It can be seen from the test results that the bimaterial interface in the deck is activated, but it is not possible to quantify the shear stresses in the interface from the measured difference in strain. However, for measurements at side A of support 1 a rough estimation from the differences in average strain between the measure points indicates a shear stress of 2 –3 MPa in the bimaterial interface, which seems reasonable. Furthermore, no signs of failure in the interface can be found, either from the test result or from the examination of the beam after testing. Hence, the composite action seems to be intact.

As for the compression strains and stresses in the CRC, the highest compressive stress in the CRC from the FE analyses was 137 MPa (for the load 2 x 430 kN) while the highest measured (calculated from the strain) was 121.4 MPa, although not in coinciding points.

4.6 Concluding remarks

The study of bonded interfaces showed that the most suitable surface treatment for the bimaterial interface in the bridge deck seemed to be the special GFRP surface with a "peel-ply" to be removed prior to casting (surface H). It is easy to perform, possesses a sufficient capacity and has a low distortion among its test results.

For the epoxy jointed CRC surfaces, the alternative with the smooth formwork surfaces being sand-blasted before jointing (surface I) appears to be the simplest and probably the best alternative.

The prototype beam exhibited a satisfying structural behaviour during the load test, and reasonable agreement with the predictions from the FE analyses. Despite the somewhat unexpected failure mode – delamination of the GFRP plate in the deck induced by a repaired damage – the beam behaved in accordance with what was foreseen during the loading. Thus, in all essentials the load test is believed to have been successful and, apart from not reaching the expected failure mode, the aims of the test were achieved. Since the test is unique, the results cannot be statistically verified, and further testing will be necessary in the forthcoming work before decisive recommendations about the performance of the beam can be made.

In summary, the investigations performed – although not covering all details – indicate that the bridge concept can be realised from a technical structural point of view. In addition, the industrial characteristics proposed will ensure efficient production and operation of the bridge. The economic aspects, however, show that there is a need for large production series and decreasing costs for FRP relative to currently conventional construction materials, in order to make the bridge concept competitive in today's market. On the other hand, if life-cycle costs and benefits such as short construction time and improved working environment are taken into account the prospective for the bridge concept looks much better.

In the continuation of the work, there is still much to be done before the first prototype bridge can be erected. Hence, detailed design and a set of appropriate tests will be the next steps to take.

5. General discussion and conclusions

5.1 General discussion

The main questions to be answered by the work presented in this thesis were stated in section 1.2, these were:

(1) Can an industrial process be one possible solution to the deficiencies faced in bridge construction?
(2) What are the most effective ways to create more efficient bridge construction and to enhance developments?
(3) How can structural engineering and industrial bridge engineering contribute to realising new industrial bridge concepts?

The answer to first question is undoubtedly yes, but it is conditioned to the important fact that the incentives and motivation of the participants must be large enough as to outweigh the alternatives, e.g. to sit still.

The second question is perhaps a bit more difficult to answer in a straightforward way. But it is at least possible to state that an industrial process of bridge construction is among the most effective ways to enhance efficiency and development in bridge construction.

The third question is best answered by good examples where structural and industrial engineering have had a large influence in creating competitive bridge concepts.

Signs of the importance of the research area can be noticed every day, in discussions within the construction business as well as in the society. Recently, the Swedish Government highlighted the issue in its budget proposition, see PROP. 2007/08:100 (2008), where it were stated that the proposed infrastructural funding will be complemented with suggestions on how rectify the low efficiency and the lack of productivity development in the infrastructure sector. Hence, the message of more infrastructures for the money is issued from the top management of the country.

One significant finding during the course of this research is the insight of the vast importance of the three cornerstones of industrial construction – process, production and productivity development - and the necessity of a thoroughly integration by means of using a comprehensive ITC system throughout the process, compare Figure 3. If it would be possible to focus on and develop these three areas, while at the same time integrating them by ITC, then the objective of a sustainable industrial bridge construction process would seem to be achievable.

The two studies of structural developments enclosed in this thesis have their basis on different development levels, as been discussed. The joint study forms an example of development on the detailed level, while the feasibility study of the *i-bridge* concept is performed on the conceptual level or the system level. However, both serve the purpose of creating examples of structural developments to enhance the efficiency in industrial bridge concepts. This is an important mission for researchers, but it is an even more important task for the construction business to implement new findings and to put the theory into practise.

The issue of who is to bring about the changes and to be the driving part in terms of developments and implementation of the new processes seems a key issue. Hence, a combination of the two approaches – market-pull and technology-push – seems to be the best solution in order to get onwards. Additionally, a step-wise evolution seems recommendable, where the incentives and much of the framework for the overall process is presented by the public commissioners, while the specific ideas and the new products are developed by the industry.

Regarding the incentives for motivating the participants to invest in development of a new industrial process of bridge construction and new bridge concepts there seems to be two major factors to consider. Firstly, the market volumes need to be large enough as to allow the establishment of several competitors. This means that the borderlines for the market might need to be widened beyond the country borders. Secondly, the possibilities to keep and maintain the benefits from the investment put into development must be ensured. Hence, the protection of immaterial rights plays an important role in this aspect.

5. 2 Conclusions

It can be concluded that an industrial process of bridge construction could be one of the solutions to the problems faced in bridge construction. This is however only possible if all participants are encouraged and motivated to cooperate and undertake the large developments needed. In turn, this motivation needs to be based on economic terms where the benefits are shared among the participants and not least the customer.

The vast importance of the design issues is recognised as a prerequisite for the transformation into an industrial bridge construction process to take place. Furthermore, industrial bridge engineering, defined as an approach emphasising the huge significance of structural engineering in combination with industrial matters, seems likely to have a large impact on this transformation.

Much work is still needed before a new industrial bridge process can be arrived at. However, due to the multidisciplinary features of this work, it seems unfeasible to bring about the changes with ad-hook engagements. There is a need of a more holistic view, since it remains most probable that a massive effort with multiple contributions from the different disciplines is needed. Thus, future research should aim at initialising large multidisciplinary research projects in the field of industrial construction, linking many researchers from different areas together in striving for a mutual goal of creating an industrial construction process. Although the co-ordination would be a delicate and difficult task, such projects are needed to achieve significant developments.

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Paper I

Explorations of Different Means to Achieve an Industrial Process for Building Bridges; part I – implications out of the current process

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Explorations of different means to achieve an industrial process for building bridges; part I – implications out of the current process

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ABSTRACT

In the ongoing multidisciplinary research at Chalmers University of Technology, different means of creating an industrial process for building bridges have been investigated, as a response to the urgent need of development in bridge construction. Out of the basis of structural design, the emphasis in this paper is to establish the foundations for achieving such a new process, basically through analysis of the current bridge construction process, as well as to determine the required improvements and provide a framework for a process model and for the future research. The main focus of the continued work is on investigating techniques and materials developments from a design viewpoint (e.g. concrete structures), in addition to include studies of construction characteristics such as production methods, etc. Hence both product and process development are emphasised.

Key words: concrete bridges, concrete concepts, industrial, process, design and construction, materials development.

1. GENERAL INTRODUCTION

The increasing interest in productivity development as a response to the lack of efficiency in the construction industry has highlighted the need of further research on how to make substantial improvements. Of particular interest for bridges are the benefits that could be extracted from an integrated industrial process of design and construction. In this respect, the great complexity and fragmentation of the construction process are one of the main issues that have to be addressed, not least for concrete bridges. The application of new or approved techniques, materials developments, methods of design and analysis, as well as construction methods are other important areas to be investigated. Recent development in the field of design and construction of structures has often aimed at utilising new or approved materials, for example the latest developments of concrete. The increasing use of information and communication technology (ICT) as well as more advanced computer-based analysis and simulation models are also strong ongoing trends. The major key to the construction concepts of tomorrow will be to combine these factors into an industrial process. Many suggestions on how to solve the problems in construction and improve efficiency have been presented, for example by Koskela [1], Warszawski [2], Sarja [3] and Gibb [4] and some interesting perspectives and opportunities for the next decades are given by Flanagan et al. [5].

However, the studies so far have usually not been carried out in the context of the construction process for bridges, although e.g. fib Bulletin No.9 [6] offer a study of conditions in bridge construction in an international perspective. Moreover, for bridge construction no one seems to have taken the overall approach of emphasising the development of an industrial process that takes account of all participants. Due to its multidisciplinary features, different parts of the process have been covered only separately, without essential linking. Thus, the question arises of how to achieve such a process and whether it really could solve the problems in bridge construction and encourage developments.

The purpose of the research presented in this paper is to lay the foundation for the continued research and to advance towards a new construction process by studying emergent applicable materials developments, techniques, design and construction methods to be adopted in an industrial building process for bridges, primarily concrete bridges. The work, involving many disciplines, focuses primarily on design viewpoints, but also concerns other crucial aspects such as production, economics and quality, while the results should be increased knowledge and suggestions applicable in industrial bridge construction. However, since the main objective is more efficient and rational construction of bridges, and thus includes linking of all its different parts, the point of departure for the research must be the process as a whole. The present paper is intended to fill this need by formulating the problem and defining the framework in which the research is to be carried out, as well as to evaluate directions with the highest potential for improvement.

This article has been divided into two parts, were this first part is devoted to comments and analysis of the current bridge construction process in comparison with developments in direction of a new industrial process. In the second part, the emphasis is turned towards problems and opportunities in an industrial context, were the underlying driving forces acting in favour of an industrial process are investigated and the necessary prerequisites are evaluated. Suggestions of possible solutions to problems are discussed. In addition, some priorities and conclusions about the continued work are summarised. The articles are organised to enable independently reading of each part.

2. A DIFFERENT APPROACH ON HOW TO CONSTRUCT BRIDGES

In this chapter, discussion of the underlying forces driving industrial developments for bridge construction is emphasised, also encapsulating the research problem in a condensed form. The background of this research project is more thoroughly described in Harryson & Gylltoft [7], but a short description in a picture and a few words can be found in Figure 1. The research has a focus on short- and medium-span road bridges, especially those where concrete is included.

2.1 On the agenda of industrial bridge construction

Industrial production in general is no longer equated with mass production; the emphasis has shifted towards a focus on creation of customer value. This should enable a widespread adoption of industrial concepts for bridge construction, since the ideas of mass production were never suited to bridges. Although such trends can be noticed for large bridges, few applications for minor and medium-size bridges have evolved, and this is particularly evident in Swedish bridge construction.

Of course the term ‘industrial construction’ is a bit awkward since it ought to imply in general that the construction industry – being an industry – should already be involved in industrial activities, and thus the need of industrialisation should not arise. This is the reason why it is better to use the term ‘industrial’ to imply contemporary solutions instead of ‘industrialised’, which seems more related to transforming the traditionally craft-based methods into something modern.

A major hindrance to development of new industrial bridge concepts is the prevalent market situation, with e.g. an extensive involvement of the society. The government rises funding for bridges, allowances of construction is complicated and handled by the authorities and commissioners of the same administrations act as public clients on behalf of the direct end-users, i.e. the drivers who use the bridge. Consequently the customer relationship is complex. This can cause confusion in respect of who is to benefit from enhanced customer value, resulting in a market that currently seems to act on lowest price basis solely. Thus no strong market force to pull demands has been noticed which also can be seen as one explanation to the lack of development, although a shift towards a larger commitment to development from the public clients has been noticed recently. The interaction between the industry, the public clients and administrators and the rest of the society regarding these matters has been insufficient so far. In absence of a clear ‘market-pull’, which probably would produce the best result in the short run, it seems like the strategy of ‘technology-push’ will have to be adopted. That is to provide new superior technical solutions and offer them to the clients, making them realise the increased value created. Hence, this will require a long-term commitment to research and development from e.g. the contractors, a commitment that sometimes has been noticed not to coincide with short-term demands from the market. This constitutes a large problem in introducing industrial construction processes. It seems obvious that to create a win-win situation in the long run, the best alternative would be a combination of strong forces of both ‘market-pull’ and ‘technology-push’. That is why the aim of an industrial bridge construction process must be enhanced value to both direct customers and public commissioners, in addition to be beneficial to the other participants as well. This can be interpreted as follows; the overall objective for any industrial construction process is to make the product either at a lower cost or with higher quality at the same cost, giving increased customer value. The main means of doing this is reduction of waste by practically eliminating the uncertainties and the peculiarities of construction (at the price of increased complexity though, as argued below). Other advantages that could be included are more efficient and rational construction, leading to shorter time of construction; improved employee performance, due to better working conditions; better use of resources from a public economic point of view. Further, a comprehensive process approach will provide better possibilities to predict and reduce the environmental effects of construction, to take sustainability aspects into account, and to foresee and reduce the need for maintenance.

Figure 1. One of the reasons why developments are needed – a common sight from a Swedish bridge construction site today.

The new technical solutions can origin from wide range of developments, e.g. in materials, design and construction methods etc. For example there is a large span of developments in concrete features that could be applicable. Both off-site and on-site construction can be emphasised in different concepts. It has to be kept in mind though, that technical improvements never can solve everything, hence they have to be governed by a comprehensive new industrial production process as well. However, the most important reason why contemporary concepts of industrial bridge construction have much better prerequisites and possibilities to succeed than their forerunners is, of course, the continuous rapid pace of development in information and communication technologies (ICT), along with an emergent understanding of how to use and benefit from this computerisation. The impact of further progress in ICT can hardly be overestimated, making it a prerequisite to successful implementation of an industrial process for bridge construction. ICT will provide the cement that ties the different pieces together and it will be increasingly employed to develop a more efficient and rational process. Moreover, it is essential to find solutions in which all participants can work in an integrated manner in order to avoid losing information on the way. One gradually more common solution to this problem is the support of a database throughout the whole process, also allowing the information to be reused in other projects. The current high technological level in general, and knowledge in construction management, also adds to the enhanced possibilities for success of contemporary industrial bridge construction concepts.

The drawback of industrial construction, as both Koskela [1] and Warszawski [2] conclude, is that it increases the complexity of the process. Koskela refers to the increased complexity in prefabrication (e.g. precast concrete concepts): a plurality of production locations (factory and site) causes longer flow and greater variability as well as need for coordinating the different stages; an increased amount of design must be done earlier due to prefabrication lead times, resulting in incomplete and changing orders; the error correction cycle is longer, and tolerances for dimensional accuracy are lower; thus the cost of increased non-value-adding activities often exceeds the benefits to be gained from prefabrication. It can be argued, however, that such processes are obviously far from being optimised to reap the full benefits from an industrial process, and must be regarded as yet another example of partially industrial concepts with unsatisfactory performance, e.g. industrialisation imposed on a traditional process. It is important, though, to address this deficiency in a deep and thorough manner, so that the advantages of the process strongly outweigh the drawbacks.

2.2 Perspectives on industrial bridge construction

Both researchers and practitioners have been tempted to commence work on realisation of the industrial ideas in construction ever since the massive success of other industries became a fact, although the early efforts as well as many other attempts in this area never became any competitive advantages. From the literature, the main features connected with industrial construction are identifiable (compare Löfgren & Gylltoft [8], especially for in-situ cast concrete) as standardisation, modularisation, prefabrication or off-site fabrication as well as on-site fabrication, pre-assembly, mechanisation, automation, and the use of different building systems; most of these features comply also for bridge construction. Many definitions connected with industrial construction can be found in the literature: for example, Gibb [4] defines off-site fabrication in the fullest sense as a change of the process emphasis for the project, from construction to manufacture and installation.

A brief look at these attributes from the public viewpoint immediately summons associations with large uniform, straight-in-line, dull prefabricated concrete structures. Moreover, from the modest use of prefabricated concrete elements in Swedish bridge construction, the experience of the past is that manufacturers often have neglected to provide an appropriate quality in their products. These are heavy burdens from the past to be rectified so that the client can see the actual customer value from current and future industrial concepts. Regarding the previously unsatisfactory performance of industrial construction, Gibb [4] concludes that industrialised building techniques have not been developed incrementally on a continual basis, but rather as a sporadic evolution, and have even been totally neglected at times; the construction industry has seldom focused on industrial methods for their own sake. Summing it up similarly, Sarja [3] states that the construction process and project management methods have undergone sparse industrialisation when compared to industrial production in other areas. According to Warszawski [2], the main reason for the relative lack of success for industrial construction is to be found in the absence of a system approach to construction and its efficient management. Koskela [1] refers to the previous unsatisfactory success of industrialisation as a result of deficient consideration of contemporary approaches (e.g. flow management and value management) in the theory of production.

Directing the discussion towards the specific area of industrial bridge construction, a rapid development in bridge construction commenced in Europe as a response to the vast reconstruction efforts following the war, see e.g. fib Bulletin No.9 [6] and Murillo [9]. Several bridge concepts such as the balanced cantilever, incremental launching, segmental bridges, span-by-span, progressive placement, and for larger spans cable-stayed bridges, were developed, and were accompanied by massive research efforts in industry, especially when imperfections were encountered. Also temporary bridges (similar in many ways to industrial concepts) developed very fast at this time. Since Sweden was spared from involvement in the war, much of this progress was by-passed, but most bridge techniques used today originate from these mid-century developments.

An interesting question is to what extent current bridge construction applies industrial ideas, and what developments have taken place in recent decades. In an international perspective industrial systems of prefabricated concrete elements are frequently used for minor and medium span bridges. Overviews of different concrete bridge concepts is frequently found in the literature, e.g. Muller [10], Rossner [11], Martinez y Cabrera [12] and Sundquist [13]. A significant development in prefabricated concrete bridges during the last fifteen years is inferred by Calavera [14]. The continually competitive concrete-steel composite bridge concepts have a high industrial degree (see e.g. Nakai [15]), especially for the steel girder parts, while the concrete deck slab is often traditionally constructed *in situ*. Much information on bridge design and construction can also be found in fib Bulletin No.9 [6]. Furthermore, an international outlook reveals several other currently used concepts of industrial bridge construction for large bridges.

The most evident trend is gigantism (especially when the medium for transportation is water), i.e. to manufacture as large and completed parts as possible – within transportation capacities – in a protected and highly industrial facility (sometimes far away from the construction site) and subsequently installing them on site. Some examples of this are the Store Baelt Western Bridge (e.g. Kjeldgaard & Fries [16]), the Öresund Bridge (e.g. Sorensson & Thorsen [17]), the Second Severn crossing (e.g. Gibb [4]) and the Öresund Tunnel (e.g. Spreng et al. [18]).

In summary, the disadvantages of the described concepts are that they can be economical only for large bridges, or they need to be repeated many times before they become profitable, or their degree of automation is too low. In many cases where the bridge market is rather limited or closed, these disadvantages become very evident. This is one of reasons why the development of new techniques and methods for bridge construction in Sweden has not been very progressive during recent decades, especially for concrete bridges. Most bridges are traditionally cast *in situ*, involving a massive use of manpower and craft-based techniques; e.g. pre-cast concrete elements are not commonly used. The overall impression is striking, when visiting a bridge construction site today, that no major changes have occurred, compare for example Figure 1. Essentially the same observations could have been made twenty or thirty years ago. Furthermore, most Swedish bridges are procured as design-and-build contracts. Despite the advantages expected from this form of procurement (compare e.g. Murillo [9]), the anticipated encouragement of developments and innovations has failed to appear, being counteracted by for example detailed prescriptions in design codes and traditional common practice. Beneficial from this form of procurement is that it mostly has provided a smooth construction stage with a minimum of claims. The conclusion is that the described situation creates substantial difficulties for new approaches to construction in gaining a foothold on the market. But reduced uncertainties due to an industrial process are likely to facilitate more appropriate methods of risk management, thus these difficulties can be accounted for. For example, competition on equal conditions could be achieved if an overall project insurance fee, including different concepts (traditional and industrial) as well as different bidders (i.e. different combinations of contractors and designers), were taken into account when evaluating the tendering phase, as argued in fib Bulletin No.9 [6].

2.3 Implications for the bridge-building process

It must be clearly stated, as been indicated above, that industrial bridge construction is not a single concept but rather a way of thinking and organising an industrial process with a very wide range of applications and technical solutions, ultimately resulting in many competing concepts of industrial construction of bridges. The advantage of concrete in aspects of industrial construction is obvious, both for the materials developments and out of design reasons as well as with regard to production matters. Concrete has an immense variability resulting in very flexible solutions, e.g. highly industrial concepts with a very diversified customisation of the products. However, to reach such benefits, it is incredibly important to adopt a comprehensive view to construction. For example, just moving the production indoors (e.g. prefabricated concrete) does not necessarily bring about improvements in efficiency; to be able to extract the excellence, all parts of the process must be designed for optimum performance. Hence the emphasis on the industrial bridge process must be to allow maximum efficiency through simplicity and repetition, with the customer as the focal point. As has been mentioned, the equation of industrial production with mass production, although still a common view and previously a fact, is no longer valid. Besides pure mass production, there is also mass customisation with a diversity of interchangeable features, as well as one-of-a-kind products being totally customised. The latter methods, suitable for bridge construction, can only be realised through a system approach to the entire process with a thorough interface management, thus physical, managerial and contractual interfaces are concerned (Gibb[4]).

The strategy to develop an industrial bridge construction process is basically to merge all appropriate contemporary means into a smoothly running entirety, since most necessary components or ideas on how to obtain them already exist. This implies interdisciplinary considerations in fields ranging from organisational and managerial issues to technological and production methodologies. It also calls for substantial benchmarking and transfer of technology, since there are many industries from which lessons can be learned. On the other hand, the development is counteracted by the short term thinking of the market. Hence, due to the prevailing state of conservatism in the construction industry, the strategy to implement the industrial process currently seems to lie in achieving a competitive concept that produces products solely at a lower cost, temporarily disregarding the potential of other added customer value created. However, a feature of the implementation strategy must be to interact with the customers. An interest from the public client about increased performance has recently been noticed and demands for enhanced value creation will thus release the full competitive benefits from the industrial bridge concepts.

2.4 Research issues

On the basis of the discussions of the driving forces, and given the description of the problems above (compare also part II of the article), some obvious questions arise. What can be done to straighten out the situation and to encourage development in bridge construction? Could an industrial process solve the problems and improve the conditions? These main questions lead to several sub-issues:

- Which are the advantages that the different participants want to gain from an improvement of the bridge-building process?
- How can these advantages be gained?
- What demands will have to be fulfilled in gaining them?
- What priorities can be established between different kinds of improvements, and how can one distinguish between ways of gaining these advantages?

The solutions to these sub-issues, which are the objective of this paper, are essential for directing the future work in order to answer the main questions. Will concrete be competitive in this perspective?

2.5 Research approach

An appropriate start would be analysis of the process, requiring some type of tool or model. In seeking such a tool, the theory of the TFV concept, visualised in Figure 2 (see Koskela [1]), was evaluated and thought to provide an interesting framework in this respect, especially if an overall approach could be taken. Thus, the emphasis was on applying the theories to the process as a whole, although the original intention of the TFV theory was rather to explain the production stages of the process. Hence, in this emergent application the TFV concept will serve as a theoretical foundation for the studies, linking their parts together. The general idea of this concept is the integration of the three persistently occurring views on manufacturing in general, into a comprehensive theory for production, since the three views complement each other. These views are:

- *Transformation* (T); i.e. the traditional view of manufacturing as transformation of input into output. Here the emphasis is to realise the transformation as efficiently as possible by decomposing the production into tasks and minimising the cost of each task.
- *Flow* (F); which means that the production is regarded as a flow through the process. The main principle is to reduce the share of non-value-adding activities (waste), leading to compressed lead time, reduced variability, simplicity, increased transparency and increased flexibility. Just-In-Time (JIT) and lean production are methods of the F type.
- *Value generation* (V); whose main principle is to improve customer value by ensuring that all requirements are captured, ensuring the flow-down of customer requirements, certifying the capability of the production system, and measuring value. This is the origin of the quality movement and other customer-oriented methods.

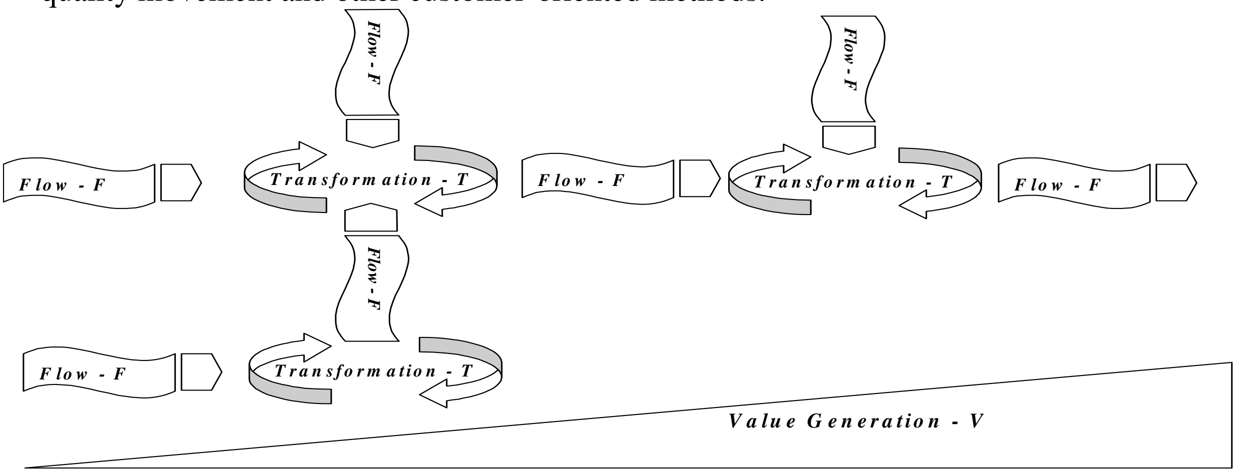

Figure 2. Generalised illustration of the TFV-scheme.

Basically, the production is looked upon as a flow with transformation parts, focusing on creating customer value through the whole process, compare Figure 2. The centre of attention in the analysis is to determine the impacts on behalf of the different views in each part of the bridge construction process. This will enable conclusions to be drawn about problems and potential needs for improvement in the process. The analysis of the bridge construction process is conducted in Chapter 3.

3. EXPLORING THE BRIDGE CONSTRUCTION PROCESS
3.1 Characteristics of the process

Compared to many other industries the general construction process shows a substantially higher degree of complexity, the large number of participants being one reason for its fragmentation. Another major problem is the common division between design and construction, which confuses customer relationships; moreover, the peculiarities of the construction business add to the complexity (compare also part II of this article). Products of the construction industry are stationary with an anticipated long service life, whereas the production facilities are movable, i.e. totally contradictory conditions compared to most other industries. When compared to the construction process in general (for buildings etc.), the infrastructure construction process is at once simpler and more complicated. It is simpler in the sense that fewer participants are involved (different trades, subcontractors etc.), but more complicated since infrastructure investments almost exclusively come from governmental funding, as been discussed. The latter leads to an even more complex customer relationship with a formal customer (e.g. the national rail or road administration) representing a somewhat diffuse real customer, e.g. the end-user, as can be seen in the process map of bridge construction in Figure 3. Furthermore, between the two are the political establishment and the parliament, transferring the needs from the real customer. This extensive involvement of society outweighs by far the increased simplicity due to fewer participants, thus adding more to the aforementioned complexity.

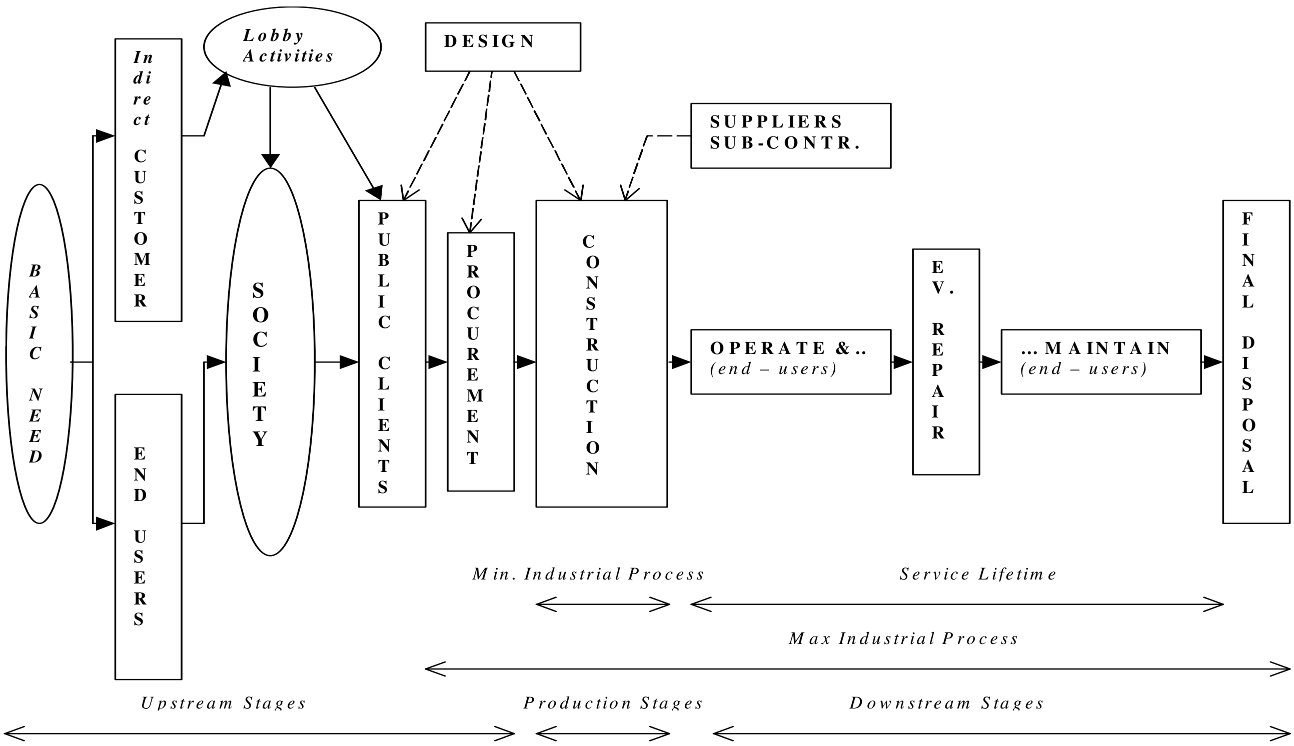

Figure 3. The conventional bridge construction process.

In addition, detailed prescriptions from the authorities put restrains on the process. In the following, 'process' will mean the whole current traditional construction process, while 'industrial process' or 'new process' stands for the suggested improved aspects of the overall process as described below.

3.2 Assessment of the entire bridge construction process on the basis of the TFV theory

As a basis for the analysis, a map of the entire bridge construction process is shown in Figure 3. This diagram was derived from empirical and individual experience, but guidance in the scope of international bridge construction processes can be found in fib Bulletin No. 9 [6]. In the analysis, each part of the process is dealt with separately and comments are presented occasionally. Details from the analysis are presented in the Appendix.

When analysing the process of bridge construction from a TFV viewpoint, it becomes very clear that the emphasis for the three different views changes as the process continues. To generalise, it seems that the changes in content for the views are correlated and occur at the same position in the process scheme, as indicated in Figure 3. This change of scope is most obvious for the view of value generation (V). In the first upstream part, the focus of this parameter is solely on gradual clarification of the needs and demands of the customer; no actual creation of value can be found in these stages. On the other hand, the next production parts, design and construction, are very closely connected with value creation. In the last downstream stages, the focus changes again, now towards checking on the fulfilment of previous demands. The same phases for the flow (F) view consist of flow of information, real flow (e.g. products, material, workforce, documents, etc.) and flow of services. A similar comparison for the transformation (T) view would imply no transformation in the upstream parts, significant transformation taking place in design and construction, and a very small degree of transformation in the downstream parts (e.g. to postpone the degradation of the structure). It is not surprising though, to find that the emphasis of the TFV theory is somewhat altered when applied to the process as a whole. Out of necessity, the following analysis must be kept at a rather superficial level; thus a further breakdown in several steps is possible, but out of place in this context. In the following some comments on the different stages are given and details of the analysis regarding the TFV theory can be found in the Appendix.

<u>Upstream stages:</u> *(also compare Appendix, Table A1)*

End-users/customers: The real customer (in contrast to the formal customer, i.e. the rail or road administration) can be divided into end-users or direct customers, i.e. the public traffic, the drivers, etc., and indirect customers, i.e. the business community, the social community (local governments, the union, etc.) and professional organisations or interest groups. The direct customer influences the process primarily through democratic elections, while the indirect customers mostly are active in considerable lobbying directed to the political establishment. Some organisations even undertake investigations to promote their own interests.

Society: By society in this context is meant the government, the parliament and other political instances as well as governmental departments and public authorities other than road and rail administrations (which are treated separately in their role as public commissioners).

Road or Rail Administration: The society grants funding to the road or rail administration. It is usually to some extent up to the administration to decide on how to divide the funding.

Procurement: Disregarding the possibility of accounting extra rewards for other parameters than the price, in reality the currently most common form of procurement is awarding the bidder with the lowest price the contract on a fixed-price basis, also for design-and-build contracts. These contractual conditions are routinely transferred from the contractor through the whole value chain. Although unusual, there has been some experimentation with other forms, e.g. functional contracts, and contracts with different forms of involvement in financing, operation, maintenance, etc. This is probably something that can be expected to increase significantly in the near future.

A development in terms of procurement is very important and beneficial for the overall process. Interesting experiences and opinions on different procurement forms in an international perspective can be found in fib Bulletin No. 9 [6]. As been mentioned, it is inferred that a system with a project-global insurance including both design and construction and paid for by the commissioner or included in the tendering offer, would be self-regulatory in terms of applying appropriate risks. Both at company and project level, with due regard to qualifications, experience etc. and the risk for each specific project, as well as between participants (e.g. avoid putting a high risk on designer whose fee only represent a few percent of the construction cost), since the insurance fee will be adjusted in accordance not only to the specific project risks, but also according to the performance of the different combinations of contractors-designers-commissioners. This system would provide great opportunities to implement new industrial bridge concepts.

<u>Production stages:</u> *(also compare Appendix, Table A2)*

Design: The design stage is usually divided into three phases, namely conceptual design, preliminary design and detailed design, each phase performed in different stages in the bridge building process. In turn, detailed design can be divided into general design (or basic design) and final design (or execution design), although this division is not often used in Sweden. A consultant contracted by the commissioner sometimes does the final design, but more often the design falls under the responsibility of the contractor, since most small and medium-size bridges are procured as design-and-build contracts, as been discussed. Hence, many conceptual considerations due to alternative solutions originate from the tendering phase. This fragmentation in design phases is unfortunate but currently necessary to keep pace with the overall process, especially in the first two phases where for example the need for coordination with the road design is sometimes overlooked. However, the isolation from other parts in the process is not beneficial and often leads to confusion in the customer perspective and to the absence of a comprehensive view. In fact, the design stage can often be compared to a black box. One possible solution is to use other forms of procurement where the contractor comes in at early stages, thus taking part directly in the conceptual discussions or to compete with different bridge concepts.

Construction: Since construction falls within the original application of the TFV theory it is apparently considered in depth by Koskela [1], but a brief summary and some complementary comments regarding bridge construction can be found in the Appendix.

<u>Downstream stages:</u> *(also compare Appendix, Table A3)*

Operation and maintenance: The longest stage in the process is closely related to compliance with previously stated requirements, and thus emphasises on minimising maintenance. Recently it has been highlighted as a very important part of the process (see e.g. Silfwerbrand & Sundquist [19] about the issue of operation and maintenance). In this stage the final user can see the results of the process and it is also the part that governs the overall cost of the project, e.g. lifetime cost, environmental effects etc. Mistakes made in earlier stages often come as surprises in the form of costly reparations.

Repair or strengthening: If this stage appears, it is usually the consequence of previous mistakes or increased live load due to heavier traffic. This part of the process is usually not foreseen, or precautions would have been taken to avoid it or prepare for it. This stage can more easily be accounted for in industrial concepts.

Decommissioning and final disposal: Flexibility in disposal is an important added customer value, i.e. different ways of reusing the remains, recycling or even possibilities to rebuild the structure or parts of it elsewhere.

3.3 The industrial bridge construction process

In this chapter some possible effects and viewpoints on an idealised industrial process in a TFV perspective are summarised.

Upstream parts: There are different degrees of industrial implementation which, in the broadest sense with other forms of procurement, could constitute an involvement in the domains of the public commissioners, as shown in Figure 3. In this case the most significant gain is a major reduction of waste (i.e. non-value-adding activities such as uncertainties due to lack of clear requirements, excessive tendering phase, etc.); therefore, the improvement is mostly in the F vein. It is of course also easier to influence the transformation that is taking place, resulting in increased efficiency in the T aspect as well. All this adds up to increased customer value in the V aspect.

Procurement: As noted earlier, it is very important to develop new contractual forms to attain a process that motivates all participants in reaching an overall optimisation of the project. It is also crucial that each participant has a clear and attainable goal, i.e. being able to produce a profit. The aim is to realise both factors simultaneously and thus reach goal congruence, which is quite essential for an industrial process. As mentioned, the degree of implementation is closely connected with the form of procurement. Procurement forms allowing an early entrance of the contractor (as the process leader) will evidently contribute to the success of industrial bridge concepts as would other procurement forms e.g. the method with project-global insurance fees, as mentioned. The obstacle of the current procurement could also be overcome, but at the price of less customer value creation and less competitive concepts. Moreover, including downstream stages in the contract increases the extent of the industrial process, although this should not affect the basic emphasis of quality for the final product, they should remain the same.

With regard to the TFV theory, the same conclusions as for the traditional process apply. For example, the possibilities of reducing waste due to multiple tendering work through other forms of procurement fall very well in line with an industrial process.

Design: An industrial process in the field of bridge construction will undoubtedly mean a different way of working in the design stage, leading to product development instead of the present unique project-based design. From this will follow a new role for the designer, upgrading him to become one of the most important persons for the success of each specific industrial concept in the long run. This is also likely to erase the borderlines between contractors and design consultants, integrating them into a process-oriented team. As mentioned, in an industrial process an integrated design is a necessity: the more of the three design phases that can be incorporated in the process, the better. It is compulsory that the detailed design phase is included, and the use of some contemporary design techniques, e.g. concurrent design, seems beneficial compared to the traditional sequential design; compare e.g.

Koskela & Ballard & Tanhuanpää [20]. Depending on the rate of industrial employment, the conceptual design and the preliminary design phases could also be integrated with other forms of procurement, thus resolving the unfortunate fragmentation of the design stage and enforcing the stress upon a comprehensive view. This would also fill the great need for construction versus design coordination, currently overlooked in many cases. Production parameters, such as workability etc., can then continuously be taken into account; hence structures from the drawing table (‘desktop products’), which are simply not possible to construct, would become a thing of the past. Also the problems in coordinating the many different design disciplines could be solved, although this problem is more accentuated when design of buildings is concerned. In conclusion, design management would become easier.

Most bridge designers act as consultants, but usually the major contractors also have design departments of their own, even though they cannot be said to be an integrated part of the production currently but mostly act like in-house consultants. Thus the stakeholder viewpoint differs somewhat between the two more on the organisational level than otherwise, but this will become even more obvious since the contemporary tendency towards erased borders between contractors and designers seems to be accentuated, as mentioned. Hence, new organisational cooperation will emerge, e.g. teamwork. Some consultants may look upon these changes as threats, but they can just as well be a great opportunity, since nothing dictates who is to own the process – it could equally be a large consultant that is the driving force behind the alteration. Furthermore, other industries employing similar system have extensive amounts of consultants in their business.

From the standpoint of the V view, the enhancement in customer value would be tremendous, chiefly because of continuous product development but also due to significant improvements in the other veins. Regarding the concept of F, integration of design leads to considerable reducetion in waste; e.g. the majority of construction peculiarities can be eliminated or accounted for. This is mainly due to better possibilities to control the design and the whole process, so that the uncertainties are significantly reduced, especially if all design phases can be included. In terms of the T view, an integrated design would lead to enormous opportunities for creating more efficient transformation tasks. Again, the influence of further developments in ICT and computerised design cannot be overestimated in this aspect, with regard to both calculations/analysis and drawings (e.g. integrating the two into one transformation task and taking other parameters into account, model based design etc.) as well as communication with other participants, e.g. coordination with road design in early stages. ICT is also necessary to facilitate the fullest sense of computer integrated construction (CIC), thus possibilities to build the bridge on forehand in the computer will further enhance the waste reduction.

Construction*:* Naturally, the great bulk of the industrial process will fall under the construction stage, since the physical completion of the project takes place here. Again, the huge anticipated impact of further evolution in the information and communication techno-logies (ICT) will be substantial and beneficial. Generally, the same conclusions as for the traditional process apply with regard to the TFV theory. However, for an industrial process there must be a major shift towards a more thorough consideration of the V and F principles, although proper attention still has to be paid to the T view. The increased customer value reached by the combination of reducing waste and improving efficiency in the transformation tasks enables the overall emphasis of the TFV concept to fit extraordinarily well for an industrial process.

On the other hand, an industrial process inevitably leads to increased complexity, especially in the F view, as mentioned. This contradiction has to be dealt with, one solution possibly being further enhancement in the use of ICT throughout the process to reduce the effects of the enlarged complexity to an acceptable level. Even if the complexity is increased, the flows in the pro-cess are known with a minimum of uncertainties, which can thus be acted upon in a continuous process-improvement cycle. Hence the advantages of the process will by far overweigh the drawbacks.

Downstream stages: The downstream parts are not likely to be included in the industrial process initially, even though procurement forms stipulating this do exist. On the other hand, the industrial concept could include additional enhancement for these aspects (e.g. considering the life cycle cost, LCC, etc.); thus the procurement will have to take this into account. Otherwise, the same conclusions as for the traditional process apply with regard to the TFV theory. Further, efforts in after-sales activities (e.g. longer guaranties, inspections, maintenance, etc.) will presumably increase as the competition eventually shifts towards delivering highest customer value.

3.4 Demands and advantages extracted from the process in view of different stakeholders

It is obvious that a certain number of demands must be raised, but also that the participants can claim many additional benefits from the new process. However, a distinction must be made concerning the driving parties of change (i.e. primarily public commissioners, contractors or even consultants) who expect to reap some predetermined advantages, and those taking part indirectly in the changes (i.e. the rest of the participants) who will attain their advantages as a consequence of the new process. Furthermore, the difference between advantages and demands is not always clear-cut; hence fulfilment of the demands sometimes also entails an advantage, and therefore it must be stressed that the advantages inferred here are complementary to the specified demands. The demands and advantages, categorised according to the original stakeholders, are presented in the Appendix, Table A4 and Table A5. They were derived from the process analysis and from the literature.

3.5 Performance specification

The requests from the different stakeholders stated in the foregoing, can be condensed into a performance specification for an industrial bridge-building process as presented in Table 1.

Table 1. Performance specification for concepts of Industrial Bridge Construction.

Basic demands: Demands that can be stated for any construction project. - Functionality, safety, serviceability and comfort criteria. - Economic and efficient construction, and delivery within time schedule. - Quality and durability aspects, lifetime assessment, maintenance reduction and environmental concern.
Advantages causing added value: Advantages (or bonus) on behalf of an industrial process. - Reducing overall construction time and cost. - Flexible process, both during construction and afterwards and no disturbance due to construction work. - Optimisation of design and an enjoyable experience of the aesthetics. - “Extras” creating goodwill, e.g. compression of time schedules, easier procurement and less - administration.

- Improved quality aiming at zero mistakes and consequently no guarantee work.
- Continuous development and improvement of products and productivity.

Complementary demands for an industrial process:

Besides the first general demands, the other examples depend to a great extent on the features of each specific concept; thus there is no general applicability.

- Reduction or elimination of wastes, uncertainties and peculiarities.
- Simple and rational construction or manufacturing, resulting in lowered construction cost.
- Easy, fast and straightforward design, integrated in the process and simplicity in aesthetic adjustments.
- Improved working conditions for employees.
- Simple and minimised work on site and all work carried out in sequence, thus avoiding later rework.
- Simple and light elements to minimise transportation, heavy lifts etc. and to provide efficient assembly.
- Open systems with standardised interfaces of components.
- For prefabricated systems, a high degree of prefabrication is aimed at, preferably with no on-site work except erection and assembly.

3.6 Framework for an industrial process

To be able to implement a new industrial process, some underlying model must be applied. The process analysis performed calls for an outline of such a model for an industrial bridge-building process, and some suggestions in this direction are presented here. The conclusions are that a framework for a process model can be derived from the analysis, also serving as a framework for the research, and it is presented in Figure 4.

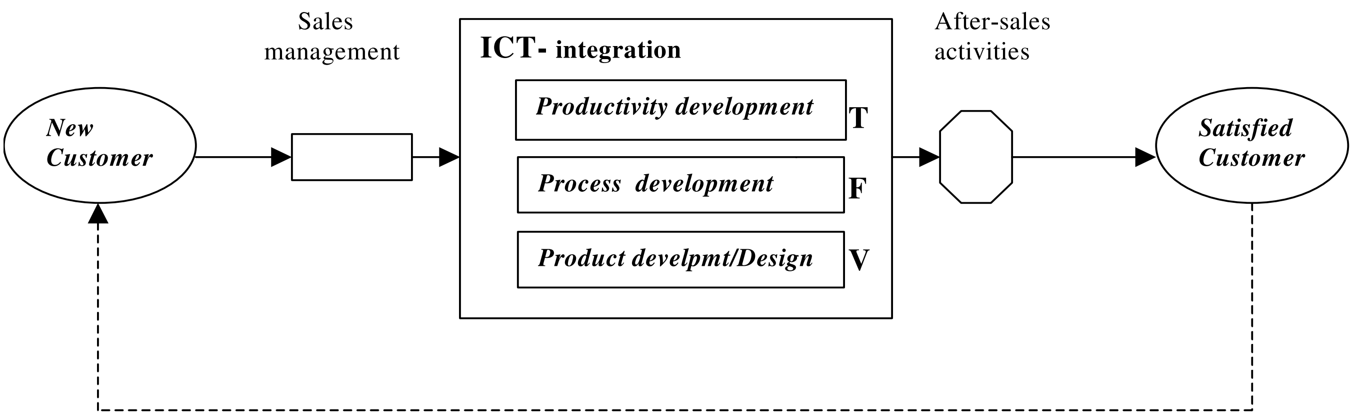

Figure 4. A model of an industrial process for bridge construction out of a managerial perspective.

From a managerial perspective the new process is looked upon as three parallel streams with regard to the TFV theory: productivity development with main emphasis on the T view, process development with main emphasis on the F view and product development/design with main emphasis on the V view, with integrated linkage between them, thus naturally, each stream will take all views into account. Sales management could for example include promotion of company specific bridge concept with brand image etc., and after-sales activities refers to creating enhanced customer value, for example through offering different additional services, such as operation and maintenance, longer guarantees or inspections, etc. It is essential to adopt a clear-cut customer/seller perspective in the new process, especially if a 'technology-push' approach seems to be necessary, as been mentioned. However, to attain a truly industrial process there is also need for another package integrating the parts and, as mentioned, this is the powerful support from ICT throughout the whole process.

It has to be kept in mind, though, that the implementation of a new process like the proposed one for industrial bridge construction is a managerial question. Without devoted support and strategic commitment from the management, no changes will occur.

3.7 Conclusions

As a summary of part one of this article, one can conclude that developments in bridge engineering is lacking behind and that there is an urgent need to promote research aiming at progress in bridge construction. In perspective of the current process for building bridges, it is evident that a development towards a new industrial process seems beneficial for all parties involved. However, in order to be able to achieve such an evolution there are many factors, research fields and research topics involved which have to be dealt with and combined in an interactive way. These questions are dealt with in the second part of this article, **'**Explorations of different means to achieve an industrial process for building bridges; part II – evaluation of contemporary strategies', were also some conclusion about the continued work are drawn.

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APPENDIX: ANALYSIS OF THE BRIDGE CONSTRUCTION PROCESS ON THE BASIS OF THE *TFV* THEORY

The Appendix contains five tables (table A1 – A5) with detailed results from the analysis conducted on the basis of the TFV theory. The tables are referred to in chapter 3.2 and chapter 3.4.

Table A1. Upstream parts of the current process in the three TFV views, compare chapter 3.2.

Basic needs of infrastructure:

V: The actual source of the V view.

F: The actual source of the F view.

T: No tangible transformations are done here.

End-users:

V: The needs and demands are further refined in this stage.

F: A large flow of information and argument takes place, mostly informal.

T: No tangible transformations are done here.

Society:

V: It falls upon society to make necessary priorities regarding the funding, thus deciding to what extent the communicated needs can be realised.

F: In the F vein there is still a large flow of information.

T: No tangible transformations are done here.

Road or Rail Administration:

V: There is still no value generation taking place, but the demands become even more detailed as they stream through this stage and funding for physical projects is crystallised.

F: There is a large internal and external flow in huge administrations, mainly information and communication with other authorities or within the organisation. Thus, great amounts of waste can be found, e.g. in pure administration such as handling of documents, checking, controlling, remittances, etc., but the main waste is lost time. Naturally, to some extent this cannot be avoided.

T: The first transformation can be recognised, the planning which is the first phase for a physical project. However, planning in early stages is closely connected with overall investigations, prognoses, financing, etc.; thus there is a lot of input to be dealt with. The output should be a conceptualisation of the project, also regarding some initial design considerations. Hence, the first connection to the early design stage, the conceptual design, can be found here, although bridges usually are very superficially treated at this stage.

Procurement:

V: Procurement is more a transfer of customer demands than an actual generation of customer value. However, the contractual condition under which the work is carried out has proved to be one of the most important factors, with a major impact on the final result and the motivation of each contracted part. Thus, it implicitly plays a significant role for the customer satisfaction as well. Nonetheless, it can also have explicit influence in special cases where an alternative solution is obtained in the tendering phase.

F: This stage consists of non-value-adding activities, the prime waste being in the large amount of contractors performing the same tendering work. Yet one obvious way of reducing this is another form of procurement where, for example, contractors can offer their products and concepts. The common case of design-build contracts often leads to sub-optimisations due to the design consultant being procured at a fixed price by the contractor with guarantees concerning the bill of masses from the tendering, thus often resulting in design with slim structures and low workability. Hence, procurement means a lot to the efficiency of the process.

T: No actual transformation takes place, unless regarding the above-mentioned case of a better alternative solution coming to the surface.

Table A2. Design and construction of current process in the three TFV views, see chapter 3.2.

Design:

V: The design stage is essential from the value generation viewpoint, V, since it is when streaming through this stage that the somewhat widespread customer needs become concretised into physical descriptions and documents of the final requirements. In other words, quantitative customer needs are turned into qualitative customer requirements. In some respects thought, detailed prescriptions from authorities counteract the value creation. It is a well-known fact that a large part of the mistakes encountered downstream in the process can be related to the design stage; thus the quality assignment must be very strong here.

F: The massive flow of information and related waste, such as uncertainties due to lack of information, wrong information etc., and control or verification has been thoroughly treated by other authors, e.g. Koskela [1]; especially the authority control of design documents is tedious in the present process. The lack of transparency is evident, particularly in the detailed design phase.

T: The transformation taking place is primarily of information into design documents. Thus, one obvious way of creating a more efficient design is further enhancement in the use of ICT throughout the design phases, for communications, documents, drawings, calculations, approval, etc. Linking of all parties can be achieved e.g. by using a project database or a product model. It is also very important to acquire and use the best knowledge as a resource in the transformation; hence knowledge management is a vital issue.

Construction:

V: This is where customer value is realised through fulfilling the wishes and requirements of the client. This is also the stage where value easily can be lost, e.g. if the importance of proper planning is not considered, disregarding the fact that systematic planning and quality go hand in hand with creation of customer value.

F: Numerous internal flows can be found, e.g. building material and products, workforce, logistics, resources (tools, machines, etc.), subcontractors and suppliers, etc. Consequently, there is also a lot of waste to be found, both physical and intangible, e.g. material waste, coordination problems, etc. As for the overall idea of continuous compression of lead time to reduce waste, it has to be borne in mind that this time can only be decreased to a certain limit without the input of a new and more efficient approach (e.g. the use of new designs or new construction methods); otherwise the quality of the final product will clearly be put at risk.

T: Multiple transformation tasks are taking place during this stage. In brief, input in the form of material, components and workforce are transformed into the finished product; thus, this stage has traditionally been very much a matter of reducing the costs in these transformation tasks. Among contractors who currently base their business primarily upon on-site concrete production, there is sometimes a tendency to exaggerate the use of *in-situ* cast concrete based on short term decisions, instead of the most efficient combination of material use, in order to keep as much as possible of the value chain within the firms.

Table A3. Downstream stages of current process in the three TFV views, compare chapter 3.2.

Operation and maintenance:

V: This stage is a matter of living up to what has been promised. There is no actual creation of value, but the significant stress is on fulfilling previous commitments, even though the formal guarantee of the contractor currently seldom reaches more than a few percent of the construction lifetime.

F: A certain amount of maintenance activities occurs, but these are likely to be predetermined and a consequence of earlier decisions.

T: An input in the form of maintenance results in achieving the predetermined lifetime.

Repair or strengthening:

TFV: Basically the same considerations as for construction above.

Decommissioning and final disposal:

TFV: Most of the considerations about the Operation and Maintenance stage, with some general modifications, are also applicable here.

*Table A4. Demands and advantages for the commissioner and the contractor, compare chapter 3.4. General demands (**D**) for any process, advantages (**A**) from an industrial process and demands specific to an industrial process (**DI**).*

Public authority commissioners:

D: The most important demands transferred from the users can be recognised as (compare Table 5):

- Functionality of the structure, safety against failure, and serviceability demands, e.g. deflection, settlements, cracking, etc., and comfort criteria such as avoidance of expansion joints and demands of continuity.
- Safety for users, i.e. traffic environment and traffic safety.
- Minimal disturbances to traffic during construction.
- Adaptability to local conditions and minimum impact on local environment including noise reduction, aesthetic compatibility, etc., and environmental aspects, including final disposal or recycling, etc.

D: Demands with a bearing on the role of administrator and maintainer are:

- Low cost on a lifetime basis, low need of maintenance and high durability.
- Easy access for planned work, easy control assessments and easy operation.

A: The advantages that public authority commissioners could reap from the new process are:

- Better quality, less repair and maintenance, as well as lower lifetime cost.
- Better possibilities for eventual future strengthening or even reuse of the bridge.
- Easier and more cost-efficient control of design and construction.

Both the commissioners and the contractor:

D: Interrelated demands that cannot be separated from both the commissioners' and the contractor's viewpoint:

- Low production cost and an efficient construction stage and delivery within time schedule.
- Quality aspects and practical environmental considerations in construction work.

A: Advantages from the new process derived from both the commissioners' and the contractor's viewpoint:

- Improvements in construction efficiency resulting in lower building costs and shorter construction time.
- Higher quality with no need of guarantee repairs.
- Better working conditions leading to improved employee performance.
- Simpler forms of procurement, reduced administration, short and predictable design stage.
- Reasons to anticipate a continuous product and productivity development.
- Closer cooperation and a process with fewer uncertainties (e.g. design, cost, time, etc.)
- Flexibility during construction and during service life.

Contractors:

D: Demands specifically related to the contractor could be specified as:

- Competitive product with high return on investments (ROI).
- Low sensitivity to unpredictable circumstances (e.g. weather conditions).
- Possibilities to protect intellectual property , i.e. the concept.
- Possibilities to achieve reliable information about future demand for new bridges.

DI: Some additional demands with regard to an industrial process are:

- A product possible to produce in a serial process, and thus minimising the impact due to advantages of scale, i.e. reducing the dependency on large volumes to carry the necessary investments.
- Trustworthy and optimised design integrated in the industrial process, and easy, predictable work on site.
- Extension of the value chains into a value network, thus empowering all participants.

A: The main advantages from an industrial process for the contractor's scope of interest are:

- Great possibilities to direct and develop the new process, thus creating good opportunities to produce a reasonable profit and a product that yields goodwill and hence creates a positive image.

- Improved employee performance, due to better working conditions.
- Better opportunities to take advantage of personal knowledge and ideas through empowerment.
- Competitive advantages, and increasing market shares on a larger and more stable market, e.g. the global market on the basis of patents.
- Possibilities to create niches with even higher ROI from ventures in R&D, especially in areas protected by patents.
- Possibilities for an efficient value network management.
- Possibilities to apply the new process in other construction areas, e.g. road construction and buildings.

*Table A5. Demands and advantages for the other participants, compare chapter 3.4; (for (**D**), (**A**) and (**DI**) compare Table A4).*

All participants:

A: The main overall advantage with regard to all users is the anticipated economic benefit that can be derived from a more efficient industrial process. This is an evident increase in customer value. Beneficial to all participants will also be better possibilities to foresee and reduce the environmental effects of the construction, enhanced sustainability in consequence of increased quality, and a significant increase in attractiveness to interest educated young people in joining the construction business.

End-users (direct customers):

D: Demands from the end-users' viewpoint are mainly concerned with functionality, safety and comfort, i.e. mostly design problems normally dealt with in design codes and standards. Another great concern, perhaps of more importance in day-to-day life, is the issue of minimising or avoiding disturbances to traffic, e.g. due to maintenance, rehabilitation and the like. Also the problem of small deficiencies in performance (such as a bump when passing the expansion joint), and the issue of aesthetics, attracts a lot of attention.

Professional organisations and interest groups (indirect customers):

D/A: Organisations generally pay attention to the functionality of the total system, thus applying the same demands on the bridges as a part of the system. The advantages that they can rely upon from the new process are more acceptance of their needs and use of the new process as an argument in their lobbying activities.

Society:

D: The demands of society basically conform to the political constraints. It is very difficult to generalise these demands since they are a product of the current opinion and political majority; thus they can shift very rapidly at times. Typically, the demands of society reflect a situation balancing the need of improved communications with environmental consciousness and economic limitations. An investment in infrastructure has to be profitable in relation to the society's dimensions.

A: The advantages that society can achieve from an industrial process are better and more efficient communications that give spin-off effects, since it is a dynamic source of activities leading to developments in society and benefits for the economy (e.g. increased competitiveness for industry, growth in enterprises, decreased unemployment and increased tax income); thus it will become easier to raise funding and to find alternative solutions for funding. Moreover, better communications are a solution to some of the problems caused by increased urbanisation. Other benefits could include greater possibilities of counteracting urbanisation and a better overall use of resources from a public economic point of view.

Designers:

D: Some general demands to be raised by the designer are:

- Visible and comprehensive methods for design management and a better coordination of the activities.
- Reducing uncertainty by achieving the right information in the right time.
- Contractual incentives enabling reasonable payment for efficient solutions that lead to lower construction costs, rather than acting on a fixed lump sum price; and to extract feedback from the production.

A: The advantages for designers from the new process are mainly a different and a more stimulating way of working in teams, thus more like other industries, which will enhance the competition for competent personnel between industries. On the other hand, the new process will attract many young people into construction. Other benefits from the new process are working as a product developer close to the production, good possibilities of continual personal life-long competence development, and possibilities of enlarging the design task to include the production process, erection, assembly, etc.

Suppliers and subcontractors:

DI: Suppliers and subcontractors represent a significant part of the value chains, although they sometimes have difficulties in influencing the process since they are considerably fragmented. Thus, their demands primarily stemming from the industrial process can be perceived as:

- Cooperation on a long-term basis, thus able to yield a reasonable profit.
- R&D commitments in joint ventures; and planning facilities that allows one to keep ahead of time.
- Participation in networks with effective value network management.

A: The anticipated advantages on behalf of the suppliers and subcontractors are better possibilities for future planning due to long-term cooperation possibly as a result of partnership, and thus possibilities of taking part in the development of products and processes. Moreover, new business opportunities are likely to occur due to outsourcing, mainly from contractors.

Paper II

Explorations of Different Means to Achieve an Industrial Process for Building Bridges;
part II – evaluation of contemporary strategies

Harryson, P.

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Explorations of different means to achieve an industrial process for building bridges; part II – evaluation of contemporary strategies

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ABSTRACT

In the ongoing multidisciplinary research at Chalmers University of Technology, different means of creating an industrial process for building bridges have been investigated, since there is an urgent need of development in bridge construction. Out of the basis of structural design, the emphasis in this paper is to establish the foundations for achieving such a new process, basically through analysis of the current bridge construction process, as well as to determine the required improvements and provide a framework for a process model and for the future research. The main focus of the continued work is on investigating techniques and materials developments from a design viewpoint, in addition to include studies of construction characteristics such as production methods, etc. Hence both product and process development are emphasised.

Key words: concrete bridges, concrete concepts, industrial, process, design and construction, materials development.

1. INTRODUCTION – PART II

This article has been divided into two parts were the first part, 'Explorations of different means to achieve an industrial process for building bridges; part I – implications out of the current process', was devoted to comments and analysis of the current bridge construction process in comparison with developments in direction of a new industrial process. A general introduction to the research topic can also be found in part I. In this second part, the emphasis is turned towards problems and opportunities in an industrial context, were the underlying driving forces acting in favour of an industrial process are investigated and the necessary prerequisites are evaluated. In addition, some priorities and conclusions about the continued work are summarised.

Some of the interacting components that can be connected to Industrial Bridge Construction are illustrated in Figure 1.

2. PERSPECTIVES ON OPPORTUNITIES AND PROBLEMS IN AN INDUSTRIAL CONTEXT

2.1 Underlying driving forces in favour of an industrial process

In light of the problems surveyed in the first part of this paper and below (see Chapter 2.2), it is important to apply a comprehensive view in discussing solutions. From a global perspective, it is a widespread opinion that the construction industry faces major changes, as is also true for the whole of society.

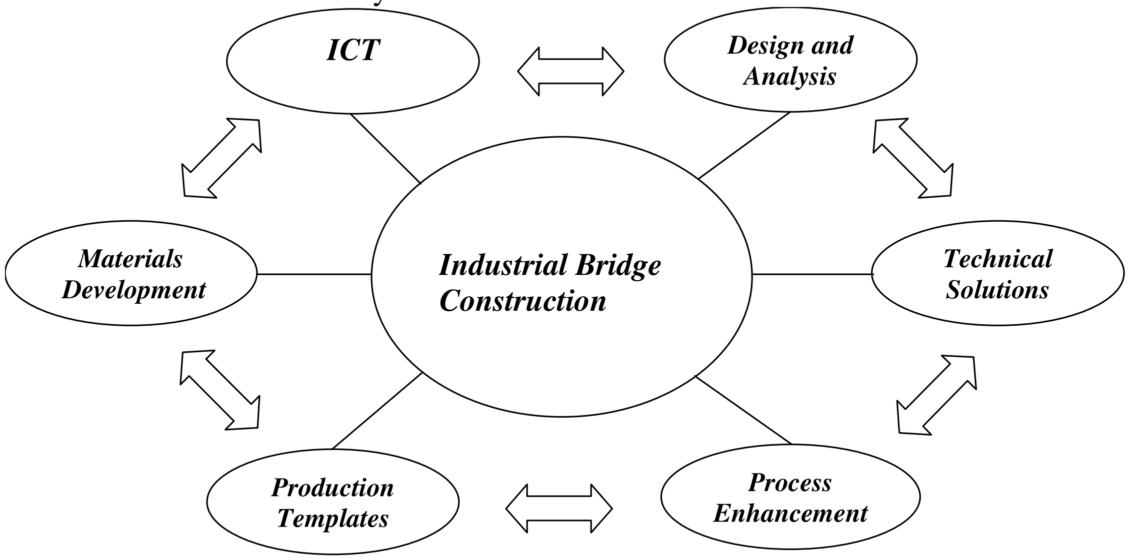

Figure 1. Some of the interacting components connected to Industrial Bridge Construction.

Flanagan et al. [1] describe the overall driver of change as social, technological, economic, environmental and political (STEEP). Demographic changes are one of the most powerful factors affecting worldwide development in a broad sense. The chief features of these changes are exceedingly high population growth in developing countries and declining growth and an elderly population in developed countries, as well as continued urbanisation. Hence, the applicable solutions will differ significantly between regions, since for example the educational level, the wage levels and the general degree of industrialisation strongly influences the development. This is somewhat illuminated by Sarja [2], who argues that it is small, mainly European countries that lead the industrialisation process of construction worldwide; in larger countries under the influence of strong building traditions, the level of industrialisation is lower than would be allowed by the general level of knowledge and industrialisation of the society

Effects of the global economy will impact significantly with its fast rate of change and many fresh opportunities of doing business (see e.g. Otley [3] for a summary of these impacts). Globalisation and the expansion of markets, causing companies to strive for world class to survive, will not spare the construction business, resulting in fewer and larger companies and many small service providers. Furthermore, a great increase in both cooperation and frenzied competition resulting in 'co-petition' will enhance the formation of strategic alliances (also with companies from outside the construction industry) and partnership with suppliers, as well as to increased specialisation. Initially, this progress will be enhanced by the need of companies to ultimately improve their revenues in order to increase their capitalisation and become attractive for investors on the market. But as the process continues, there will probably be a shift in emphasis of the driving force towards a stakeholder perspective, since other incentives than purely economic variables – such as lack of labour and attractiveness to employees (losing manual skills because workers cannot see a future in working under harsh and unsafe conditions, as well as for demographic reasons of ageing) or public confidence as a response to opinions in society – will also play a very important role, requiring companies to act accordingly.

The effects on the organisational level will also be extensive, and the managerial approach will have to comprehend a variety of issues throughout the whole organisation. There will be a specialisation on behalf of company undertakings as well as within organisations; thus integrated multidisciplinary teams, complemented by outsourced specialised services, will replace functional departments in companies. Managerial efforts must be devoted to logistics, supply chain and network management to facilitate partnership commitments. Since knowledge in the whole organisation as well as in individuals will be a major factor for competitiveness, knowledge management will be essential; hence, continuous learning on both individual and organisational levels will be vital. Otley [3] remarks that this is especially important for '*the control of knowledge-based workers where the key resource is time and the key outputs include innovation and responsiveness to customer demands*', e.g. service providers or consultants. Furthermore, reduced uncertainties due to new processes will facilitate more appropriate methods of risk management. Thus, the overall emphasis is likely to shift towards fulfilling customer needs and competing by providing added value rather than on lowest building cost solely; to seek demanding customers driving the development can become a competitive advantage. This indicates large benefits from a strong commitment to innovation throughout the organisation.

In this context a comprehensive theory for continuous strategic renewal is of vital importance for seizing the right opportunities of doing business in the global economy. The framework presented by Simons [4] is interesting in managerial respects. Simons stresses the importance of managerial support and argues in favour of maximising return-on-management (ROM) as a substitute or complement to return-on-investments (ROI) in order to enhance the outputs of value. He concludes that the process is a balancing act between the dynamics of creating value, the dynamics of strategy-making and the dynamics of human behaviour, each of them causing organisational tensions which must be coordinated to allow effective implementation and control of business strategy. Anthony & Govindarajan [5] also provide some interesting views on management control, especially stating that the central role of management control systems is to achieve goal congruence, defined thus: '*Goal congruence in a process means that actions it leads people to take in accordance with their perceived self-interest are also in the best interest of the organisation*'. Another aspect of the problem of goal congruence is notably present in many current forms of procurement in construction, namely a lack of goal congruence between the different participants. Hence, changing 'people' for 'participants' and 'organisation' for 'project' makes the definition still valid. Otley [3] puts the development of organisations into perspective: '*But although survival has evolutionary connotations, and the idea of the 'survival of the fittest' is much mooted in the rhetoric of market-based economies, we have to move beyond biological ideas of evolution. The problem with the evolutionary model of biology lies in the prolific inefficiency of nature and the relatively slow pace of random evolutionary adaptation. Thus we need to move towards the idea of evolution by design, by designing organisations that are capable of redesigning themselves.*' Moreover, Otley points to a crucial condition concerning all innovations and developments, which is particularly present in the construction industry: '*A further area lies in the encouragement of innovation and appropriate risk-taking. Most performance measurement systems encourage conservatism and 'playing it safe'. Managers need to be encouraged to identify defined areas* *within which a degree of experimentation and risk-taking might be beneficial. Too often we stifle creativity and learning by insisting upon good performance from all activities.*' And in a similar vein: '*In essence we are having to move away from hierarchical, top-down approaches to control to one where self-control, innovation and empowerment are of at least equal importance.*'

More directly addressing the construction sector, Koskela [6] in his thorough research refers to three contemporary views on production (the TFV theory, as been mentioned in part I) that must be extensively linked together and balanced also in regard to construction management resulting in task management, flow management and value management. Koskela's conclusion is that construction now faces the same situation, as did manufacturing in the early 1980s when new production developments (Just In Time, JIT, and the Quality movement) caused reevaluation of most prevalent approaches to production management. As for innovation in construction and its inherent risks, Beeby [7] suggests that the risks be reduced by innovating in relatively small steps with a consolidation after each step, and that innovations in the process of design and construction may be less risky than changes in the technology so that larger changes can be attempted. However, it must be argued firstly that it is very difficult to govern improvements and to predict which way to go; secondly, if it were possible to choose, customers probably would not allow adoption of a slower incremental by-pass, with due regard to the ever more rapid pace of changes on a global scale; and thirdly, innovations and development are urgently needed in both process and technological perspectives.

2.2 Problem statement – the basis of opportunities

Problems in construction will have to be addressed in the near future if the industry is going to fulfil its commitments to society. Paradoxically, these problems also create great opportunities for innovations. On the other hand, the insufficient acquaintance with developments and new building systems among professionals is a serious problem in itself and provides the greatest impediment to successful application in practice, as e.g. Warszawski [8] argues. There are quite extensive documentations of the problems related to the construction industry to be found in the literature. While numerous investigations have been focusing on the problems, the suggested solutions have been implemented seldom or adopted only partially. Several governmental surveys (e.g. Egan [9], SOU 2002:115 [10] and others) can be summarised into the conclusion that the main problems show great similarity regardless of country of origin. Löfgren & Gylltoft [11] offer a comprehensive review of the problems concerning construction; furthermore e.g. Koskela [6] provides some analysis of the background and the roots of the problems. In general, there are some significant sources of problems that seem to be intrusively connected and interrelated, for example the complexity and the fragmentation of the industry, the peculiarities of construction (see further Koskela [6]; especially important for bridge construction is the extensive governmental involvement as noticed in the process analysis in part I), the common division between design and construction, deficiencies in managerial coordination, and the large amount of waste and value loss that is encompassed in construction. All of these add up to the primary problem of low efficiency and profitability in the construction industry. The entrance resistance is very low, among other things leading to a large number of participants in the process (one reason of complexity in construction) and extensive competition based on lowest price. Investment in both knowledge and capital can thus be kept at a low level, and the emphasis on research and development (R&D) is negligible compared to many other industries.

However, there is really not much to be found in the literature regarding the problems in the specific sector of bridge construction (e.g. some problems in procurement etc. can be found in fib Bulletin No.9 [12] and Murillo [13] describes the problems overseas), thus some of the problems are described in the following. Although primarily referring to Swedish conditions, the description could to some extent be general applicable. To begin with, the different participants in the building process each have their own reasons and motives for acting in a certain way (as seen from the process analysis in part I of the article). Consequently, many problems are related to a lack of communication and cooperation between the parties. This also increases the fragmentation of the process, leading to each participant trying to optimise the efforts within his own limited field. Hence, no one seems to take on the responsibility of an overall approach aiming at optimising the entire process to benefit all of the participants. Here the need of strong clients being able to lead the development is obvious. Furthermore, the linkage of problems in construction seems to resemble a downward spiral loop or a vicious circle. First of all, there is a high level of competition in the matured industry of bridge construction, mainly because of the low entrance resistance. Although somewhat higher than for construction in general, still both the capital employment and the technical level needed for entrance are relatively low. Secondly, this leads to lower margins in business revenue, limiting the scope for investments in R&D. In addition, there is a large risk in venturing into R&D since there are great difficulties in assuring that advantages can be kept to benefit the investor alone. Thirdly, what follows seems to be the stagnation and rigidity of the current situation, with continued lack of improvement in efficiency and productivity aggravating the conditions. Thus, another round downward is completed. Koskela & Vrijhoef [14] also argue that the prevalent theory of construction (i.e. the transformation theory) is the main hindrance to innovations and developments in the construction sector, a statement with relevance in some respects.

In addition, some facts that promote uncertainty in the bridge construction industry are:

- A great variation in volumes from year to year. The main reason is that most infrastructure investment is subject to political decisions, although sensitivity to economic fluctuations also has some influence.
- The production series, apart from one-of-a-kind projects, are very small. Moreover, the extensive variability in geometry, aesthetics, etc., makes each bridge unique. This does not leave much economic space for heavy investments in equipment etc. for a given system; in fact, it works in favour of preserving the entrance resistance at a low level.
- The large infrastructure commissioner, mostly public authorities, seems mainly satisfied with the situation over the last decades. Thus, they have not used their huge influence as the main direct clients to change the process of bridge construction to any observable extent. Only recently, a change in this aspect can be noticed.
- The significant conservatism in the construction industry leads to problems of introducing new ideas, due to lack of confidence among the participants in the process.

On the other hand, one can argue that the great increase in uncertainty that all businesses have to cope with caused for example by the globalisation, and that project based production has become more common in general, consequently leads to an enhancement in similarity of the conditions for other industries with those for the construction sector. As the problems and uncertainties in construction by necessity seem to drive the development in a direction closer to other industries, this will also add to the increased similarity mentioned. In conclusion, this should also provide enhanced opportunities for the construction industry to apply the same approach to methods, strategies, production, etc., as in other industries. These circumstances are illustrated in Figure 2.

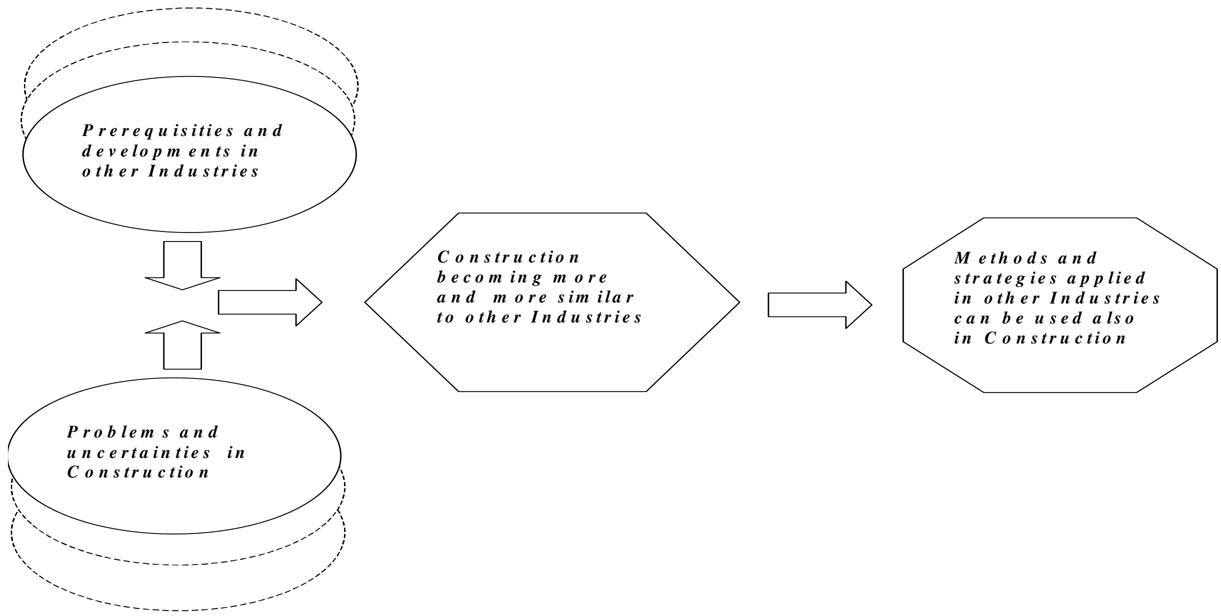

Figure 2. Problems – the base of opportunity.

3. EMERGENT STRATEGIES ON HOW TO ATTAIN AN INDUSTRIAL PROCESS

3.1 The basis for foundation of industrial bridge construction

As we have seen, there are many different ways leading towards an industrial process for bridge construction. In this brief summary, emphasis is laid chiefly on overall factors influencing the creation of a new process. In order to reach sustainable industrial development in bridge construction, some fundamental demands will have to be fulfilled. There must be suggestions on how to organise the industrial construction process, since the current process is not particularly suited for industrial purposes. Some kind of consistent theory of production will have to support the industrial ideas. There also have to be some basic ideas of industrial concepts, utilising appropriate developments in materials, design and production methods, etc. In addition, there is need for a comprehensive interconnection of all the parts, namely by means of information and communication technology (ICT), as have been discussed (compare part I of this paper).

Moreover, effects of globalisation and expanding markets is likely to result in considerations about demands for rapid development of new products and continual improvement in cost efficiency becoming very important, and the only way to secure the competitiveness of a concept over a period of time is to ensure continuous development and improvement. This is one of the principles from the Quality movement (see for example Frid [15] for an introducetion). In terms of the process, this will mean constant process development. Consequently, a continual flow of ideas is needed. It has to be clearly stressed that the industrial process is merely an infrastructure for an efficient production of bridges, linking the different parts involved. Thus one vital task is to offer guidance and hosting for good ideas and innovations.

Industrial concepts of bridge construction are possible only through intensive correlation and co-operation between different disciplines, as has been pointed out. Accordingly, there will be aspects bearing on the organisational or managerial domain as well as aspects stressing the technical domain, but there will be no aspect with emphasis in just one of the domains. The cornerstones of industrial concepts of bridge construction can be identified as the three P's –

process development, productivity development and product development – forming a progressive, continuous circle integrated by ICT, as visualised in Figure 3.

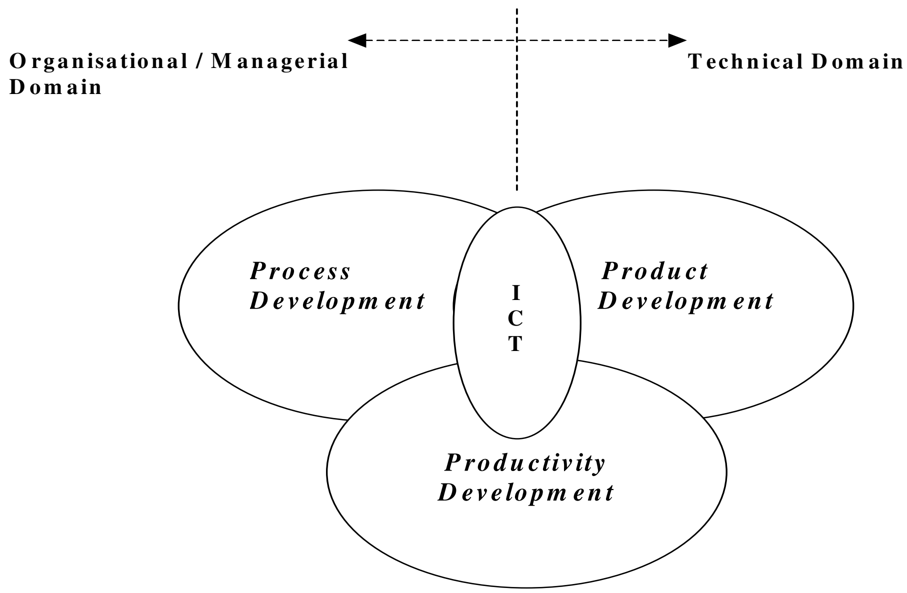

Figure 3. The cornerstones of Industrial Bridge Construction, the three P's.

To begin implementing industrial ideas, it seems necessary to suggest an appropriate sketch of a suitable process to channel the new ideas. For a start, the process is likely to contain the core of construction – the production and the design stages. In a way the two other P´s, productivity and product development are embraced with the process, since the process will have take all activities and all participants into account as well as constituting the infrastructure to support and govern the actual production. The main emphasis in the process development is flow management (the F view in the TFV philosophy, compare Koskela [6] and part I of this article), aiming at eliminating waste and no-value-adding activities. The implementation of an industrial process will provide one part of the immediate competitiveness of the bridge concept, while to ensure continuous process development will enhance the performance and govern the competetiveness in the long run. As the industry and the customers become more familiar with industrial concepts, the borderlines of the process are likely to be extended. For example, it is possible that sales management will be somewhat integrated into the domain of the commissioner, allowing early contributions in the planning phase of infrastructure projects. At the other end of the process, it is possible that after-sales management will develop to provide different services in the 'operation and maintenance' stage. An outline of the process from a managerial perspective has been presented in part I of this article (part I, Figure 4).

Product development can be seen as a continuous loop of optimisation and refinement of the features of the product, responding to feedback of deficiencies from the whole process, as well as the initial design of new products. The aim of the improvements is to enhance the performance of the product, which covers a wide range, from aspects such as easier production, simpler assembly or jointing on-site, emphasising more efficient construction, to features such as a more appealing layout of the design or improvements in durability, more directly relevant to the customer relationship. This is probably one of few opportunities to ensure competitiveness over a period of time. Hence, emphasis is mainly on value creation and value management (the V view in the TFV philosophy) in product development.

By productivity development, as a cornerstone in this context, is meant merely the production issue of how to attain more efficient production, for example by rationalising prefabricated production in factories or the execution on-site. Thus, the main emphasis of productivity development is in the transformation vein, the T view of the TFV concept – to execute the decomposed transformation tasks in the most efficient way – although, of course, the two other P's contribute in rationalising these tasks. It can be said that productivity development is the ultimate goal, i.e. more efficient construction of bridges, while the two other P's tend to be the means of fulfilling this goal, by developing the products, smoothing the way and providing the necessary tools for governing progress in the right direction.

Thus, all three cornerstones are closely linked and appropriate attention must be paid to each of the P's in order to reap the full benefits of industrial concepts. For example, a new product with excellent performance has small chances of becoming a success if it is not produced efficiently and if it is not guided by a rational process all the way to the market. Similarly, an excellent production technique is hardly relevant if the product is undeveloped and unwanted on the market, or if the process fails in providing a market connection. Thus, it can be said that the three P's have their 'centres of gravity' in somewhat different domains, as been noted above (compare Figure 3). While process development seems to rely more on the organisational/managerial domain, the emphasis of product development lies in the technical domain, and productivity development tends to stress both domains about equally. It has to be remembered, though, that as in the case of balancing the three P's, efforts in the two domains must be thoroughly balanced as well.

The subsequent work builds upon demands raised by the different participants in the process with the overall goal of satisfying the customers, i.e. real customers and society represented by public clients. As a part of this, one aim is to suggest possible solutions to the problems the industry is facing (see above and e.g. SOU 2002:115 [10]). It is evident that the driving participant in terms of developments, changes and implementation of the new process must be the industry providing and utilising the products, i.e. primarily public commissioners, contractors or even consultants. Hence, other participants in the process will have to be convinced and realise the benefits.

3.2 Emerging technical solutions

Current research and development are yielding a large amount of promising results, and the great challenge is to combine the right features that can materialise into emerging technical solutions. In this context, a few very interesting areas with a bearing on bridge construction will be discussed.

In the field of materials science, there are many applications that would fit into an industrial concept. Such applications in the field of concrete are high-performance concrete or ultra-high-strength concrete, self-compacting concrete, fiber reinforced concrete using different fiber materials (e.g. see Ay [16] and Guerrini & Rosati [17], about research on steel fiber reinforced high-performance concrete for bridge structures), lightweight aggregate concrete and combinations of these. In this project, studies on an innovative solution for structural joints between prefabricated concrete elements have been conducted; see Harryson [18]. This result from materials development in combination with research in structural engineering can be seen in

Figure 4 (the joint before casting). The concrete in the joint is a steel fiber reinforced high strength concrete called Compact Reinforced Composite (CRC).

Other materials developments to be combined with concrete (since a bridge are likely to always contain concrete in some way) are high-strength steel and the exciting progress in fiber reinforced polymer composites. Also new developments in wood technologies could be of interest (especially in composite action with e.g. concrete), but they will have to be excluded since wood will not withstand the excessive lifetime assessments under severe environmental conditions that are normally used for bridges. Moreover, Flanagan et al. [1] explore some other emergent materials developments, such as ceramics, shape memory alloys, biomimetics ('the abstraction of good design from nature') and nanotechnology (ultimately to construct material 'bottom-up', i.e. from the level of atoms or molecules). It is quite possible that some of these advances will be applicable also in bridge construction. Further, recovered techniques such as concrete-filled steel pipes or externally prestressed concrete, and new developments in adhesives, new joints between components, and many more that could be mentioned are, of course, highly applicable in the context of industrial bridge concepts. Another exciting tendency is development of smart structures, i.e. structures with embedded systems (e.g. microchips, optical fibre) that can communicate to simplify construction or to facilitate continuous monitoring, etc.

Other developments, such as in manufacturing (e.g. automation, robotics), transportation (e.g. road-rail solutions, by air etc.) as well as assembly and erection techniques (e.g. heavy lifting, jointing techniques) will also play a significant role in specific concepts; for example, in concepts with a high degree of prefabrication the transportation and erection of the components will be crucial issues. In this respect active benchmarking of other industries is extremely valuable. Developments in on-site activities such as participating formwork and efficient scaffolding could also be of interest for *in-situ* cast alternatives.

Figure 4. An example of result from combination of materials development and research in structural engineering; an innovative new joint concept for structural joints between concrete members.

A variety of different prefabricated concrete concepts, with different combinations of con-crete technologies and variation in degrees of prefabrication, will be highly competitive. Pro-vided that a proper and rational detailing is developed, especially of such essential and sensitive parts (both for constructability and durability reasons) as the construction joints, it is most likely that precast concrete will somehow be included in a successful concept of industrial construction. It is obvious that combinations with industrial *in-situ* cast parts could also produce strong alternatives.

Further development of composite concepts, such as the traditional one with the combination of steel and concrete, will continue to be competitive. But more interesting are the new innovative composite concepts that will emerge, utilising the essence of the latest materials developments. The problem in this context is not to find applications with potential, but rather to get things going at the start since new materials developments tend to be very expensive before they are commonly used in large volumes. It is necessary that the concepts that are likely to surface adopt a system approach, so that the system will include not only all parts of design and construction, but all parts of the industrial process as well

3.3 The important design issues

As mentioned, the latest materials developments are likely to lay the foundation for competitive industrial bridge concepts. However, the counterpart of these developments in structural design research seems to be lagging behind, thus becoming the bottleneck and a major hindrance in implementing the research results. There is a great need for this research in order to obtain the necessary authority approvals, and the insufficient cooperation between structural-design and materials scientists is very unfortunate. Also in the light of the problems faced with two new post-tensioned box-girder bridges in the Stockholm area in Sweden during 2002 see Hallbjörn [19], which seems to be the result of lacking knowledge in structural engineering, the importance of research into design issues cannot be stressed enough. The major question to be answered by researchers in structural design is how to design with these materials, resulting in practical design rules, especially when more than one material is concerned and composite action is accounted for. Thus, the conventional aspects of design (e.g. safety against failure, serviceability demands, fatigue) will have to be investigated. Other aspects that have been focused on recently are durability and lifetime assessments, sometimes summarised with regard to economic and environmental effects in life cycle analysis or life cycle cost.

Another very important design issue with regard to new materials developments is to find the optimal structural forms for the new material. Hence, copying the old conventional forms and cross sections used for our ordinary materials will not do; e.g. compare Keller [20]. One certain feature of a competitive future concept is to use the right material with the right structural form in the right place combined with a high structural utilisation of the materials. Moreover, research in loadings can be beneficial, especially since live load and load from constraints have such a major impact on bridges; the actual load must be correct before it can become worthwhile to optimise the structure. However, this liability falls upon the public authorities responsible for the design codes.

As far as building systems are concerned, an extremely vital issue is to develop fast, smooth and simple detailing of joints between components (inferred to be a large problem by Warszawski [8]) for example standard interfaces. As been mentioned, studies on an innovative solution for joints between prefabricated concrete elements have been conducted in this project; see Harryson [18].

Other design issues more specifically addressed to each concept are those connected with the constraints of the production, transportation, erection and assembly; solutions allowing a rapid and smooth transition through these phases (e.g. production-friendly solutions) are valuable. In addition, design of the temporary works needed, i.e. the on-site preparations, will have to be addressed.

3.4 Towards new templates for production

As been noted by Koskela & Vrijhoef [14], the construction industry has neglected or not managed to implement at least two major production templates that deeply restructured other industries – namely the mass production and lean construction (e.g. JIT). The arguments from the industry was (to some extent rightly) that the specific problems connected with the peculiarities of construction could not be overcome. However, time is now proceeding towards the inevitable point where new templates for production will be an inescapable reality. New computerised features such as e-Tags (or i-Tags) and e-Commerce will most likely entail a small revolution in the whole chain of businesses, not least in the supply chain (or supply network) management. Encouraged by the continuing enhancement in benefits from ICT, the tendencies of this industrial evolution could be similar to the suggestions in the following.

A strong improvement factor, probably becoming a prerequisite some years from now, is to perform construction in a protected environment. This means weather-protected production (indoors in industrial facilities or temporarily covered on-site work), but also provision of efficient facilities and safety precautions. It must be kept in mind that weather protection is only the means, and that it is supply of the appropriate methods that leads to increased efficiency, which is sometimes forgotten; compare Moström & Asplund & Samuelsson [21].

As for most heavy work in developed countries, the development in the construction business will not differ – machines will replace heavy manual work. This is for economic reasons (cost reduction) but also due to problems in finding employees willing to undertake these heavy tasks and for labour safety. These developments have already started with demands from authorities on labour environments as the driving force. Hence, there will be a significant increase in mechanisation of construction sites, both on-site and off-site. Further developments in machines, tools and other equipment will enhance the trend of man being replaced by machines. With the aid of powerful computer support and ICT, the automation will gain speed and industrial manufacturing in construction will use robotics extensively, especially in off-site production. The conclusion is that the production will become more high-tech and the facilities will be more like any ordinary industrial production for both on-site and off-site production.

As for the completion of structures on site, the evolving methods of erection and assembly as well as other on-site preparations will strive towards simplicity with a minimum of manual work. There will be a high degree of mechanisation, and developments in heavy lifting, mounting, jointing etc., will encompass many special machines and equipment to allow rapid installation, thus increased parts of structures (components as well as systems) is likely to be manufactured elsewhere and assembled on site. An objective will also be to complete the structure at once, thus avoiding later on-site reestablishment for complementary works. In Figure 5, an example of a common semi-industrial bridge concept is shown, a bridge with prefabricated pretensioned concrete beams and prefabricated concrete slab as lost formwork for the traffic slab. This must be regarded as an example of an 'industrialised' bridge concept, as argued before (compare Chapter 2.1 in part I).

Figure 5. A common semi-industrial bridge concept, here an example from the island of Hawaii.

4. SETTING PRIORITIES
4.1 Developments with potential

All developments presented above have large potential for improving efficiency in construction. The following main features are therefore discussed in the light of the key issues of potentiality, time horizon and difficulties in implementation.

ICT and computer science:
Developments in ICT will provide an infrastructure for the system; thus there is an urgent need for this development and it also has a very high potential. The current development is extreme-ely rapid, so the time issue should not cause problems. Such developments are also exceptional-ly easy to implement if they are user-friendly, since there are no authorities or similar agencies to give approval and the general computer consciousness is increasing irreversibly. On the ot-her hand, this calls for caution and appropriate controls to avoid drawbacks, e.g. bugs in the software products.

Structural design and materials science:
Developments in structural design and materials science are extremely important in order to proceed with new concepts of potentially high interest, not least the developments in concrete. The state of research differs depending on the material, but generally there are several materials developments ready to implement, e.g. fiber reinforced polymer composites. Usually, however, the research in design matters is lacking behind, as argued earlier, and will possibly act as a bottleneck when implementing these results. The design matters are also very important in

order to achieve the necessary authority approvals, which probably is one of the difficult stages to overcome in the implementation of different developments. In addition, the general conservatism pervading the industry will increase the time for implementation.

Process developments:
Compared to the current situation, there is a very high potential in introducing process improvements. A model of the new process seems as a prerequisite for implementing a new concept and as an example a framework for a process model has been presented in part I of this paper (part I, Figure 4). However, implementation is likely to be gradual depending on the generality of the model as well as the trust generated among the participants; hence a good start would be implements parts of the process or to find ways of verifying a process model at small scale.

Basic production template:
On-site versus off-site production is a continuously interesting issue. As far as bridges are concerned, the big achievement is likely to be in off-site fabrication, if the increased complexity due to the industrial process is firmly counteracted as been argued. There are of course opportunities to industrialise on-site fabrication to a certain extent as well (e.g. prefabricated reinforcement cages, temporary weather protection, the use of self-compacting concrete, integrated formwork, etc.), but in the context of traditional on-site construction it is possible to sense the limitations already at the start. However, there will most likely develop competitive on-site concepts (or in combination with prefabricated components), but it is argued that work on site should be minimised (even though it is unavoidable to some extent), and the potentially most interesting of the basic conceptual principles undoubtedly seems to be off-site fabrication.

Production methods and techniques:
Production methods and techniques, despite their high potential, can be established in detail only in conjunction with the basic concept or after it is decided upon. Many developments are available on the market, i.e. many applications are already implemented, so the vital work will be to combine the most competitive methods and techniques.

4.2 Priorities between developments

The conclusion from the above is that all development areas are needed for implementation of the final concepts and setting priorities are difficult. However, to accomplish the gradual improvements that are likely to take place, development in ICT seems most important to provide the infrastructure for the new process, but since research in ICT is beyond the scope of this project the developments in this area must instead be followed closely although no actual work will be done in it. It is also a high priority to commence the implementation of the new industrial process by finding ways to study or verify a process model. Of equally high priority is the issue of continuing the recent materials developments into the design area, so as to be able to introduce them on the construction site. In this respect, it is crucial to investigate those applications that are in line with a suggested concept of industrial bridge construction, for example if prefabricated concrete elements are included in the concept, the detailing will be essential to investigate – especially new solutions of structural joints between the elements, as has been mentioned. When deciding upon a concept, investigations regarding production methods etc. become interesting. For the continuation of this research, some conclusions about special project priorities have been drawn in Chapter 5.

5. CONCLUSIONS

Apparently, we are close to a major shift in the foundations of the construction industry, and this is particularly obvious in bridge construction. The results of the present and other research clearly indicate that there is an immediate need for a substantial alteration in the current construction process. Furthermore, the need spans across the whole industry: there are demands for new form of procurement, new ways of financing, new production templates, a new integrated design process, new forms of long-term cooperation, new ways of communicating, increased development of even more powerful ICT applications, attraction of more competent young people into the construction business, and also most urgently the need of developments in new techniques and methods, as well as an adoption of developments in recent materials science. The conclusion is that implementation of an appropriate industrial bridge-building process among all participants would definitely lead to changes and provide possibilities to solve many problems. If it could be made interesting and prove profitable to enhance developments, the transformation would be initiated and gradually the principles of the new paradigm will spread among all participants. Actually, the limitations of a new process are only in the minds of the participants.

The real physical limitations, though, consist of a risk of not being able to present feasible solutions, especially with regard to the previously stated problem of increased complexity or on purely technological matters. It must be emphasised that, with the conservatism that currently permeates the construction business, new concepts to be adopted into an industrial bridge-building process will have to be competitive and show profitability from the outset if they are likely to be given a reasonable chance of survival on the market. This is probably the most challenging and most difficult task to succeed with. Obviously, there is much work to be done before we arrive at new concepts and a new process ready for implementation. Given the multidisciplinary features, it is not feasible for a single researcher to accomplish this; more probably, massive efforts will result in multiple contributions from different disciplines. It would be a real progressive step to initialise a major multidisciplinary research project in the field of industrial construction, linking many researchers from different areas together to strive for a mutual goal. For the continuation of this research at Chalmers University of Technology, some conclusions about project priorities must be drawn in order to provide suggestions for the future work. The importance of the client perspective, both the direct customer (e.g. drivers etc.) and the formal customer (e.g. the national road or rail administrations), must be stressed in this aspect, that is to focus on features beneficial to the client. Therefore, the first overall emphasis will be to identify a potentially successful concept for industrial bridge construction and to apply the basic ideas outlined above to this concept. With this concept as the point of departure, the main focus will be to investigate new or approved techniques as a continuation of recent materials developments from a design viewpoint, but also to perform studies of construction characteristics such as production methods, assembly, etc. for the specific concept. The aim is also if possible, to try to find ways of studying or verifying a suitable model of the process from a client perspective, if feasible within the same bridge concept.

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Paper III

High Performance Joints Between Prefabricated Traffic Slab Elements for Industrial Bridge Construction
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High Performance Joints Between Prefabricated Traffic Slab Elements for Industrial Bridge Construction

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ABSTRACT

A study of an innovative new concept for joints between prefabricated concrete elements has been conducted. The aim has been to design a joint that is stronger than the surrounding elements, but still a very small joint that is easy and fast to perform. A special steel fiber reinforced high performance concrete is used in the joint. Several laboratory tests have been performed showing that the joint meets the demands and can be treated monolithic in design.

1 INTRODUCTION

A project dealing with the matter of building bridges in an industrial manner is currently being conducted at Chalmers University of Technology. Proceeding onwards from today's conventional bridge construction the aim of the project, "Industrial Bridge Construction", is to suggest techniques, design methods and construction methods in order to develop a more industrial building process for bridge construction. The work is carried out based on both today's conventional bridge construction and new developments for structural use. It is a multidisciplinary project involving studies of new or approved techniques, materials, methods of design and analysis and construction methods as well as studies of the construction process. The project should finally result in a practical concept of industrial bridge construction from a design point of view, but also in terms of the essential aspects of production, economics and quality. However the main focus will be on a more efficient and rational construction of bridges. A more comprehensive description of the project can be found in Harryson and Gylltoft (2001).

This paper however, will present an interesting application in the field of detailing, namely the work with developing a new innovative construction joint for prefabricated concrete elements.

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The use of prefabricated concrete elements in bridge construction is not very common in Sweden. One of the main issues is how to detail the joints between prefabricated elements. It is necessary to find a good solution to this problem in order to promote the use of precast concrete in bridges. The joints are often the weak points of the construction and also one of the most important details in order to achieve an efficient construction on site. The joints have to have at least the same general performance as the rest of the bridge, but in the same time they must be easy executable and meet the demands of a simple erection process. With a retrospective look, it is also evident that the joints have represented a durability problem. This is mostly because of bad performance and poor detailing due to the fact that their importance for the lifetime of the bridge has not been taken fully into consideration.

2 DIFFERENT TYPES OF JOINTS

The main purpose of the joints is to bring two pieces of concrete elements together and to transfer forces acting on the structure. This must be done considering the structural system used in design, and the joints can of course be of various types. There are two fundamentally different categories of joints, namely joints transferring moments and those that are not. Experience show that examples from the second category often are less durable. Joints from the first category create a continuous structural element, resulting in a monolithic behaviour of the joined elements. This category is the most demanding, but also the one that shows the best performance and economics in the long run, not least in terms of durability. The first category is therefore often prescribed by the client, as the only type of joint allowed when precast concrete elements are concerned. There are several types of joints from this first category that are more or less suitable for concepts of prefabricated concrete element. For example the joints can be of conventional type meaning an ordinary wide in-situ cast joint with looped bars protruding from each element. They can also be dry joint if constantly compressed and the elements are match cast. Other types can be glued e.g. with epoxy, but these joints are normally not intended for structural use. There are also several examples of wet joints and the investigated joint is among them.

3 THE NEW JOINT CONCEPT

3.1 General

This contribution presents a concept for wet joints; using a steel fibre reinforced high performance concrete called CRC. In this particular application, a bridge concept initiated by the construction company NCC, Strängbetong (a producer of prefabricated concrete elements), the Swedish Institute of Steel Construction (SBI) and the consultant Scandiaconsult, the joint will be used between prefabricated traffic slab elements in a composite bridge with steel girders. However it is believed that the joint can of course be widely applicable in prefabricated concrete structures. The first joints used within the bridge concept were of conventional type, and even though several different types of joints were tried, none of them showed any good performance in aspects of efficiency, workability and economics.

The prefabricated deck slabs are normally precast with the same length as the full width of the bridge and an element width of less than two meters. Accordingly, there will be a considerable amount of joints for a normal bridge and with conventional joints a significant part of the bridge would in fact be cast in-situ. Furthermore, the mounting of the prefabricated slab elements on site will be complicated when conventional joints are used, as they involve looped bars as well as a large quantity of transverse reinforcement in the joint

In order to improve the performance of the bridge concept and the joints, Strängbetong and the cement producer Aalborg Portland initiated the development of this new type of joint. The aim is to design a joint that is stronger than the surrounding concrete elements and makes the concrete continuous, but still a very small joint that is easy and fast to perform. A comparison of the conventional joint and the new joint is shown in figure 1.

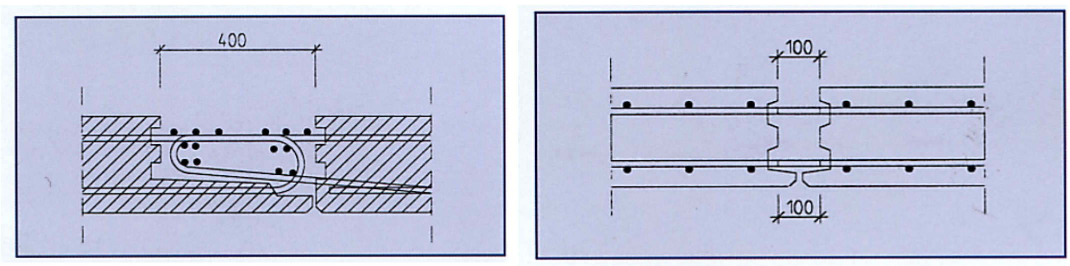

Figure 1. Comparison of joints between prefabricated deck slabs; the conventional joint (left) and the new joint (right).

3.2 Design Concept

The width of the new joint is only about 100 mm, approximately one fourth of the conventional joint, with a very simple arrangement of the reinforcement to be spliced. Only straight bars are used and a transverse bar is simply placed on top of the reinforcing bars sticking out from the elements. Despite the small dimensions, the joint is fully moment resisting and over-strong compared to the surrounding elements. The assembly and cast on site are also easy. Figure 1 is also showing the protruding lips in the new concept that are used rather than putting moulds under the joints. The thickness of the lip is reduced to a minimum in order to offer as much CRC cover as possible. A sealing strip of rubber is placed in the conjunction of the two lips to avoid leakage of concrete when casting the joint.

In order to achieve these exceptionally good joint properties a special high performance concrete is used to fill the joints.

3.3 CRC – Compact Reinforced Composite

The special concrete used is a steel fibre reinforced high performance concrete called CRC, which stands for Compact Reinforced Composite. CRC is developed by

Aalborg Portland, Denmark. A broad documentation of material properties for instance durability, fire resistance, fatigue etc. has been supplied for CRC, see Aarup and Karlsen (2000). Many research projects have been carried out, among others resulting in a design formula concerning the bond length for achieving full anchorage of the rebar. Fatigue in form of a number of pullout tests has also been done, see Nielsen (1993) and Jensen (1999).

CRC has a very high compressive strength, normally about 150 MPa, but much higher values can be reached. The fiber content is very high, typically about 6% by volume, or more than 450 kg of steel fibers per m^3 of concrete, which is the amount used in the joints. These factors provide CRC with exceptional bond properties. In addition CRC has good adhesion to previously cast concrete. Thus a very suitable application for CRC is in small and simple joints between prefabricated concrete elements. This application has been tested in slab-to-slab connections in buildings. Examples of other applications are in tunnel linings, in repair and for very thin architectural components in buildings, see further examples in Jensen (1999). The tested joint is the first application for bridges.

CRC has a high early strength, about 110 MPa after three days. This is sometimes considered in design, for instance when a reduction in construction time is desirable. This means that the capacity of the joint will exceed that of the surrounding elements later on.

Due to the small dimensions of the joint only small amounts of CRC are needed. Thus a prepared dry mortar mix called CRC Joint Cast is used, which means that water is the only additive that will have to be used on site. This special product allows for a very simple mixing of the concrete. The performance of the casting is also a very simple procedure. The CRC fills the joint easily and levels smoothly. Therefore the joint can be cast using conventional methods with a very good result.

4 LABORATORY TESTS

Several tests have been performed at the laboratory of Chalmers University of Technology. They include static tests of flexural bending moments and shear forces as well as fatigue tests. The tests have been conducted for a broad variation in the geometry of the joint, in order to find the best solution.

The studies of the new joint concept started with a M.Sc. thesis see Broo and Broo (1997). The first tests where conducted both for bending and shear capacity. The initial shear tests on 3 specimens gave results that were very satisfactory and the shear capacity of the joint was about 40% higher compared with the theoretically calculated values. It was decided that no further static shear tests were needed. The results from the first bending tests, also on 3 specimens, were not acceptable. Anchorage failures in terms of pullouts were obtained before the ultimate strength of the rebar was reached, one reason for this being that not enough CRC cover was provided. Improvements of joint geometry in order to increase the bending capacity and avoid pullout of rebars were selected as the main subject for further investigations. It was also decided to study the influence on the joint capacity from small variations of joint geometry. The layout of the new joint can be seen in figure 2.

New series of bending tests, more thoroughly described in Harryson (1999), was thus started aiming at optimization of the joint geometry with respect to flexural bending capacity. Two factors were thought to be crucial in order to be successful in these attempts. Firstly the applied concrete cover of CRC would have to be sufficient. Secondly a transverse bar placed in the joint was thought to even the stresses over the joint and to avoid pullouts.

Figure 2. The joint before casting with the reinforcement to be spliced and the transverse bars on top.

In order to convince the authorities to allow the joint system to be used for bridges, it is crucial to establish proper documentation of a satisfactory fatigue performance as well as to verify the performance under static loading. Out of this reason another series of tests was done to investigate the behaviour under fatigue. An extensive description of these tests can be found in Harryson (2000). Conclusions from the static test series resulted in an optimized joint geometry, this led to the design chosen for the fatigue test series. This featured a lap length of 100 mm, which is the same as the width of joint, two 8 mm transverse rebars in the joint and 20 mm concrete cover of CRC. Even though 15 mm CRC cover seemed to be sufficient according to the static tests, a cover 20 mm was preserved in order to facilitate some extra tolerance and allow proper compaction around the bottom rebars.

4.1 Static Tests

4.1.1 General

The test series consisted of 10 specimens with five different geometry's and were carried out on beams similar to the actual traffic slabs, also compare Harryson (1999). Jointing was done at Chalmers by putting two prefabricated elements opposite each other and casting the joint with CRC Jointcast using a normal poker vibrator. The joint before casting is shown in figure 2 above.

The prefabricated elements were produced at a factory of Strängbetong in Veddinge, Sweden. The compressive strength at 28 days was 73 MPa measured on cubes and 57 MPa measured on cylinders. The yield stress of the 16 mm reinforcing bars was 564 MPa and it failed at approximately 660 MPa. The compressive strength of the CRC at 28 days was 198 MPa measured on cubes and 150 MPa measured on cylinders.

The shear keys have been kept as for the conventional joint even though it probably could have been left out. This was done in order not to have to worry about the smooth surfaces for the prefabricated elements to be joined, but also to achieve some additional embedment length for the protruding reinforcing bars to be spliced. Because the specimens were tested with a positive flexural moment, the splicing of the bottom reinforcement was the most critical part and the subject of the tests. If the loading were to be reversed, i.e. loaded with a negative moment, it would be a more favorable situation because the top reinforcement is better embedded in CRC. A transverse bar would probably have to be put in connection with the top bars in this case though. The height of the tested specimens is 260 mm, the width is 440 mm and 16 mm bars are used for the reinforcement sticking out. In figure 3 the setup and the loading arrangement is shown.

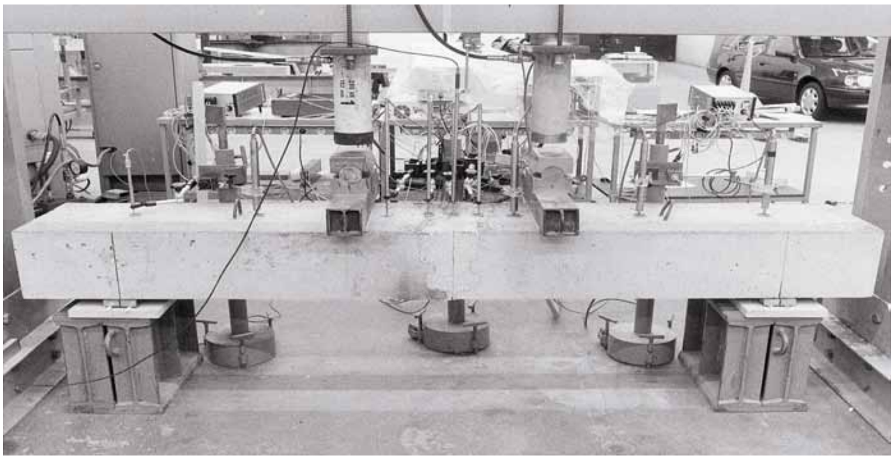

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Figure 3. Setup for the static loading test. The loading is a symmetric two point load. The specimens span 2000 mm and the space between the loading points is 600 mm. Principally the same setup is used for series UP1 and UP2 in the fatigue test.

4.1.2 Performance

The loading in the tests was controlled by the mid span deflection. Thus it was easy to observe the crack formation and the post failure behavior could also be conveniently followed. In total 8 strain gauges per specimen were attached to the reinforcing bars to be spliced (two gauges per bar), one at each side of the interface between the CRC and the ordinary concrete. Vertical deflection was gauged by in total 9 displacement transducers per element, which can be seen in figure 3.

4.1.3 Results

The results show that in the cases of bending failure, the failure always occurred outside the joint and there were no cracks within the joint. All failures (also the pullouts) occurred after yielding of the reinforcement. Most specimens achieved an ordinary bending failure, with crushing of the concrete in the compression zone. For the specimens without transverse reinforcement, specimens 7 and 8, the lowest failure load of all the tests were achieved. The failure was the result of pullouts of rebars, leading to that these two specimens show a brittle behavior at failure. Also the specimens with reduced lap length and reduced width of the joint, specimens 3 and 6, show a lower failure load. These two specimens failed in different modes, one with a somewhat more ductile behavior and ordinary bending failure and one with brittle behavior being the result of pullouts. This shows that 80 mm is close to the critical lap length that can be used and also how sensitive the joint is to variations in the geometry parameters, which is natural regarding the small dimensions of the joint. The results and a specification of the tested specimens are shown in table 1.

Spec. no.	Width of joint (mm)	Bottom Cover (mm)	Trans-verse bars	Crack load (kN)	Yield load (kN)	Max. load (kN)	Max. deflection (mm)	Type of failure
1	100	20	ø8	19.2	59.5	79.8	47.7	B
2	100	20	ø10	19.9	61.8	84.9	47.9	B
3	80	20	ø8	18.4	61.8	81.6	41.3	B*/
4	100	15	ø8	20.0	60.0	82.9	49.1	B
5	100	15	ø8	19.3	61.3	83.9	45.0	B
6	80	20	ø8	13.5	63.8	78.6	30.5	A
7	100	20	None	18.7	61.9	77.9	30.7	A
8	100	20	None	18.0	61.6	76.7	29.0	A
9	100	20	ø8	17.9	59.8	82.9	51.6	B
10	100	20	ø10	18.1	61.0	86.9	51.6	B

Table 1. Results from the static tests. Lap length is the same as the width of joint. The given load is one of the point loads (the values must be doubled for the total load). Bending failure is specified by index B and anchorage failure (pullouts) is specified by index A. By */ is meant a somewhat less ductile behavior in the post peak range.

4.2 Fatigue Tests

4.2.1 General

The test series contains six specimens with three different configurations and two different
set-ups. A more thorough description of the tests can be found in Harryson (2000). The loading used in the tests was calculated based on the Swedish code for bridges (Bro94). This was done to verify that the specimens could withstand at least 400,000 load-cycles with the maximum allowable reinforcement stress range without failure since this is the design values for similarly reinforced in-situ cast slabs. Final failure was supposed to come outside the joint area. According to Bro94 the maximum stress range for fatigue is to be based on a point load set to 90 kN acting on a area of 200 x 600 mm. The maximum stress range allowed for reinforcement is given in the

Swedish concrete code (BBK94) as 216 MPa / γ_n for 400,000 cycles. The partial safety factor γ_n is set to 1.2 for safety class 3 that is applicable for Swedish bridge design.

The two test series UP1 and UP2 was performed on similar specimens as in the static tests. The goal for these test series was to verify that the joints had sufficient strength in pure bending to withstand the conditions described above with some margin. The difference between the two series is that UP1 has medium reinforcement content while UP2 has high reinforcement content. The stress range of the reinforcement was chosen to about 1.15 times the nominal value from the code, resulting in a stress range of 250 MPa. The maximum load was set to give a flexural bending moment somewhere close to 70% of the ultimate strength in bending, which is believed to be higher than the actual maximum stress levels for fatigue in bridge slabs. The minimum load was then calculated to achieve the stress range selected above.

The primarily aim with test series UP3 was to evaluate what influence the joint would have on the fatigue behaviour and capacity in the direction of the main bending moment. Thus the two specimens in this series are oriented in the transversal bridge direction, i.e. spanning the gap between the steel beams, and the joint is cast along the elements. Since the issue of shear force transfer under cyclic loading in combination with bending was another object of interest, the joint was placed with an eccentric position in the specimens. This was done in order to achieve shear load transfer in the joint from the symmetrically placed point load in the middle of the span. The maximum load was set to 1.5 times the fatigue point load of 90 kN resulting in a maximum load of 135 kN on the loading area 200 x 600 mm. The distance between the supports in combination with the minimum load was chosen to reach a reinforcement stress range approximately in the same level as above. The set-up of the UP3 series can be seen in figure 4.

If possible, the aim of all test series was to maintain the cyclic loading until about 800,000 cycles where reached. This mean the double amount of cycles compared to what is prescribed in the codes. If the specimen sustained this loading it were to be loaded static to failure.

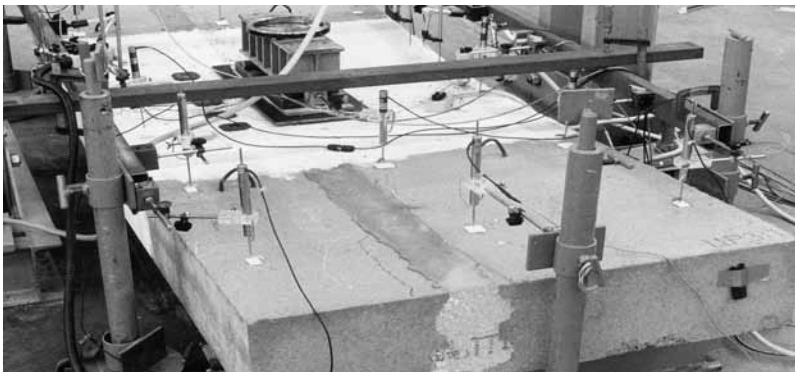

Figure 4. Setup for the fatigue test of series UP3, with the eccentric longitudinal joint and the centric loading simulating a wheel pressure.

As for the prior tests, the elements were produced at the same factory of Strängbetong. The compressive strength at 28 days was 80 MPa measured on cubes. When testing the last specimens approximately a month later, the compressive strength had reached 64 MPa measured on cylinders. The 16 mm reinforcing bars were the same type as were used in the static tests. Casting of the joints were done at Chalmers similar to the previous test series. The compressive strength of the CRC at 28 days was 200 MPa measured on cubes. At the time of the last testing the value had increased to 214 MPa.

4.2.2 Performance

The three different types of specimens tested in the three test series are shown in table 2 and the load parameters are shown in table 3. Each of the test series contained two specimens. For UP1 in total 8 strain gauges per specimen were attached to the reinforcing bars (two gauges per bar, one at each side of the interface). The number of strain gauges for UP2 increases to 12 per element. For each of the UP3 specimen in total 18 strain gauges is used, 8 attached to the bars to be spliced in the center of the span (two gauges per bar, one at each side of the interface), 8 attached to the longitudinal bars (one per bar of the main reinforcement) in the elements and 2 gauges were attached to the 8 mm bars in the joint (one on each bar) in mid span. Vertical deflection was gauged by in total 14 displacement transducers per specimen for UP1 and UP2. For UP3 this number increases to 18 transducers per element. The final static loading for those specimens subjected to this, were principally performed in the same way as previously described.

The first ten load cycles were done slowly to make it possible to follow the crack development. After this the load was applied with a frequency of approximately 5 Hz. The load cycles were interrupted at predestinated intervals when a slow cycle was performed during which data was recorded. This also enabled documentation of the progress in crack formation. The first bending cracks appeared in the beginning of the first slow load cycle. As the cycles continued, the number and width of cracks increased though there were no cracks observed in the CRC joint.

Specimen series	Length (mm)	Width (mm)	Height (mm)	No. of spliced rebars
UP1	2600	440	260	2
UP2	2600	480	260	3
UP3	3520	1200	260	8

Table 2. Description of the specimens for the fatigue tests. UP1 and UP2 are similar to the specimens of the static tests. For UP3 the joint runs along the specimens. In UP1 the spacing of the16 mm rebars are 220 mm. For UP2 the spacing is 160 mm and for UP3 the spacing is 150 mm.

Specimen series	Span (mm)	Max. load (kN)	Min. Load (kN)	Max. rebar stress (MPa)	Min. rebar stress (MPa)	Stress range (MPa)
UP1	2000	68.0	13.0	323.1	73.4	249.7
UP2	2000	94.0	13.0	300.4	50.7	249.7
UP3	2880	135.0	24.0	282.8	68.1	214.7

Table 3. Load parameters for fatigue tests, theoretically calculated values.

4.2.3 Results

Results of the fatigue test series are shown in table 4. In those cases where failure had not occurred after 800,000 cycles the specimen is loaded static to failure and this failure load can also be seen in table 4. For UP1 specimens the reinforcement broke in a normal type of fatigue failure in bending outside the joint. For the UP2 series one specimen (UP2:1) sustained more than 800,000 load cycles without reaching fatigue failure after which it was loaded static to failure. This specimen was accidentally loaded with a very high load before the test started, which is believed to be favorable with regard to fatigue.

Spec. type	Spec. no.	Failure type	No. of cycles	Crack load (kN)	Yield load (kN)	Failure load (kN)	Max. deflection (mm)
UP1	UP1:1	F/B	423000	22	--	--	--
	UP1:2	F/B	506000	20	--	--	--
UP2	UP2:1	S/B	820000	--	201	249	43.4
	UP2:2	F/A	382000	30	--	--	--
UP3	UP3:1	S/B	802000	69	318	373	49.9
	UP3:2	S/B	800000	80	322	385	56.3

Table 4. Results of the fatigue tests. Index F/B indicates fatigue failure in bending, index F/A specifies fatigue failure in anchorage and index S/B indicates failure under static loading after reaching more than 800,000 load cycles (deflection and load, other than crack load, can only be shown for these specimens).

The other specimen in this series (UP2:2) failed in anchorage before the aimed 400,000 load cycles was reached. This particular specimen had not been aligned properly when casting the joint, so the beam became oblique introducing torsion loads. This caused uneven distribution of stresses, leading to premature failure of one of the corner bars. The CRC cover for this bar was reduced due to the misalignment and also because all corner bars in this series (without consideration) had been placed to close to the outer side of the formwork. The resulting insufficient cover of CRC is most likely the major cause of the anchorage failure. The other two bars broke in normal fatigue bending failure without being pulled out. This specimen is also the only one that failed in the interface between CRC and conventional concrete, which occurred due to the pullout of the corner bar.

As can be seen from the tables, the static failure load for specimen UP2:1, was more than double the maximum load applied in the fatigue test and the difference increases for the UP3 test series. This is one of the reasons why these specimens were persistent for more than 800,000 load cycles without coming to fatigue failure.

It was obvious in the UP3 test series that the longitudinal joint caused a significant stiffening of the specimens. Although this is not considered to cause any problems, it will have to be further investigated.

5 CONLUSIONS

With sufficient lap length of the reinforcement within the joint, the joint will be over-strong when compared to the adjoining prefabricated elements due to the exceptional bond properties and higher compressive and tensile strength of the fiber reinforced matrix. Thus continuous structural elements can be created using precast concrete connected by this new joint, while treated as a monolithic member in design.

Another conclusion is that the new joint concept should show substantial improvements in efficiency, economics and workability.

The tests also show, that due to the small dimensions of the joint, it is rather sensitive to lack in tolerances. Thus a great deal of attention will have to be paid to quality control at the construction site when using this type of joint concept.

For one of the specimens in the fatigue tests, UP2:2, an anchorage failure in terms of a pullout was observed. However, since the failure occurred in just one of the corner bars with insufficient CRC cover while the other bars performed very well, this can be disregarded from when proper attention has been rewarded to this problem.

The eventual effects of stiffening due to joints in primary direction will have to be evaluated, but this is not believed to cause any problems.

The final conclusion is that the joint is applicable in this context. Thus it would be preferable to perform a full-scale pilot project as the next step. FEM-analysis is also currently being conducted to match the tests and to generalise the results. This is done in order to achieve a deeper understanding of the mechanical behaviour for the concept and will also enable the establishment of eventual theoretical models for use in design. After the analysis eventual need of further laboratory tests will be evaluated. As a result of the investigations, a design formula for the joint will have to be developed in order to make the joint compatible with the Swedish bridge code. This will also require description of performance and the essential material properties, in order not to exclude all other concretes beside the CRC Joint Cast.

6 ACKNOWLEDGEMENT

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Paper IV

High Performance Joints for Concrete Bridge Applications
Harryson, P.
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High Performance Joints for Concrete Bridge Applications

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Summary

A new concept for high performance connections between prefabricated concrete elements in bridges has been investigated at Chalmers University of Technology, Sweden. The goal has been to design a joint that makes the surrounding elements continuous, but still a small joint that is easy and fast to produce, thus rendering the joint extremely suitable for use in industrial bridge concepts. A special steel fibre-reinforced, high performance concrete is used in the joint. Several laboratory tests have been conducted as well as finite element (FE) analysis, confirming very satisfactory performance of the joint concept. A brief presentation of the concept and the laboratory test is given, with evaluations of the finite element analysis.

Introduction

The growing concern with productivity improvement as a reaction to the low efficiency in the construction industry has highlighted the need for further research to make significant progress. Of special interest for bridges are the benefits that could be extracted from an industrial process of construction. In this respect, new innovative applications in the field of detailing have great potential for rationalising the construction of bridges, especially when the use of prefabricated concrete elements is included. One of the main issues is how to detail the structural connections between the elements, as inferred in [1]. Since the joints are among the most important details determining the efficiency of construction on site, it is necessary to find a good solution to this problem in order to enhance the use of industrial bridge concepts including e.g. precast concrete, which is not very widespread for bridges in Sweden. Furthermore, the connections between elements often govern the overall structural behaviour of the system.

Structural joints between concrete elements constitute a large problem, as concluded in e.g. [2]. In Model Code 1990 [3] the necessity of easy and quick execution of jointing operations on site is pointed out, and some general guidelines for design of structural joints between prefabricated concrete elements are provided. In retrospect, it is also obvious that the joints often have represented a durability problem. For bridges, experience shows that conventional compression joints not transferring flexural moments often exhibit lower durability. Flexural and tensile joints transferring moments and tensile forces are more demanding to execute, but they display better performance and economics in the long run, not least in terms of durability. However, current types of joints in bridges have seldom performed well in all respects of interest for efficient bridge construction. Thus, one of the major issues is to provide a joint technology with the best possible performance in regard to efficiency, workability and economics, in addition to the ordinary design and durability demands.

The ongoing work presented in this paper is intended to find a solution to the above problems by developing a new concept for moment-stiff high performance joints between prefabricated concrete elements. Several laboratory tests have been conducted [4] as well as FE analysis, showing that the jointed elements can be treated as monolithic members in design. However several items need attention before the proposed concept can be adopted in practice, and one of the primary aims among further work is to conduct extended FE analysis.

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A New Concept for Structural Joints

Design Concept

The concept comprises wet joints, using a steel fibre-reinforced high performance concrete called Compact Reinforced Composite (CRC). The aim has been to design a small joint which is easy and fast to perform, and which makes the surrounding elements continuous, thus rendering the joint extremely suitable for use in industrial bridge concepts. In this particular initial application, the joint will be used between prefabricated deck slab elements in a composite bridge with steel girders. However, it is assumed that the joint can be widely applied elsewhere in prefabricated concrete structures. A comparison between one type of conventional joint and the new connection is shown in *Fig. 1*. There is a substantial number of joints in a normal bridge, so a considerable part of the bridge will actually be cast in situ when conventional joints are used. Furthermore, conventional connections complicate the mounting of the prefabricated slab elements on-site, as they usually involve a very difficult reinforcing arrangement, e.g. looped bars and a large quantity of transverse reinforcement in the joint.

The new joint is only 100 mm wide with an uncomplicated arrangement of the reinforcement to be spliced. Only straight bars are used and transverse bars is simply placed on top of the protruding reinforcing bars from the elements; thus the assembly and casting on site are also very simple procedures. Despite the small dimensions, the joint is fully moment resisting and stronger than the surrounding elements. The thickness of the protruding lips is reduced to a minimum in order to offer as much CRC cover as possible, and a sealing strip of rubber or similar material is placed in the conjunction to avoid leakage when casting the joint.

However, due to the small dimensions, quality control on site becomes an es-

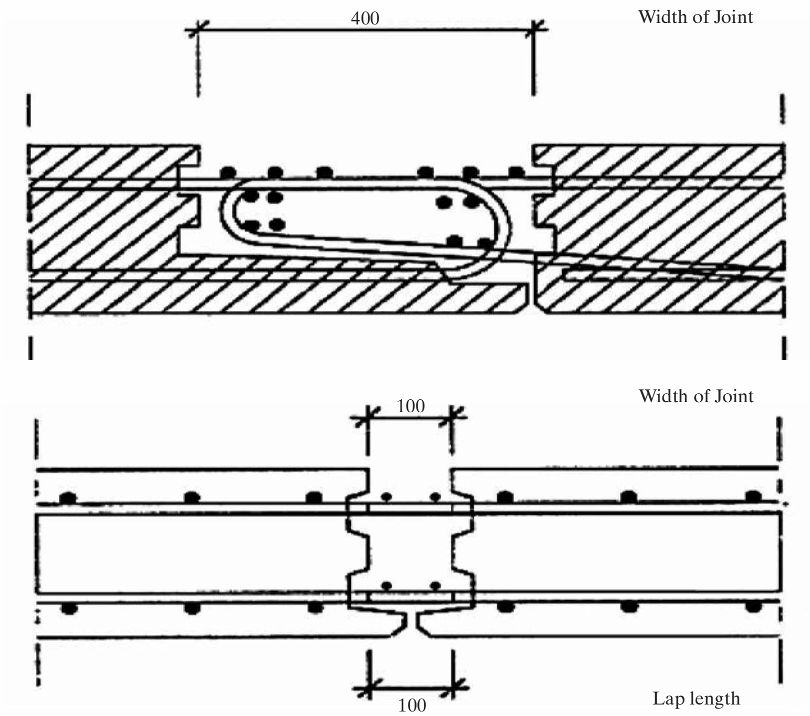


Fig. 1: Comparison of joints between prefabricated deck slabs: a conventional joint and new joint

sential issue to address tolerances and workmanship. This must be conducted in a practical manner in order to not reduce the benefits of the concept. Pictures showing the layout of the new joint concept can be seen in *Fig. 2*.

Compact Reinforced Composite

In order to achieve exceptionally good joint properties, a special high performance concrete is used, called Compact Reinforced Composite (CRC). This silica-fume-based concrete is characterized by tightly packed fine and ultra-fine particles in combination with steel fibre reinforcement. The water/binder ratio is about 0.16 and the silica fume content is 20–25%. Quartz sand with particle diameters up to 4 mm is used as aggregate. The compressive strength of CRC is very high, normally about 150 MPa, but much higher values can be reached. The fibre content for joint applications is typically about 6% by volume, or more than 450 kg of steel fibres per m^3 of concrete, which was the amount for concrete used in the joints. These features provide CRC with exceptional properties of bond to reinforcement and, in addition, with good adhesion to previously cast concrete; thus it is very suitable for joint applications. In [5] a description of the material as well as further examples of applications can be found.

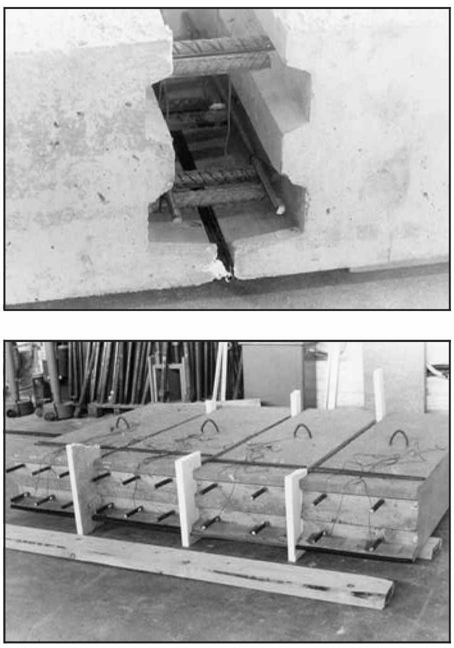


Fig. 2: Above is joint before casting with reinforcement to be spliced and transverse bars on top. Below are four halves of specimens before jointing

Because of the low water to binder ratio there are moderate shrinkage effects and for the test specimens these effects are negligible. However, the effects need to be further evaluated for each specific application when CRC is cast on-site, e.g. for the composite bridge concept with steel girders.

Due to the small dimensions of the joint, only small amounts of CRC are needed. Hence a specially prepared dry mortar mix is used, which means that water is the only additive on-site, allowing very simple mixing of the concrete. Execution of the casting is also simplified; the CRC fills the joint without difficulty and levels smoothly, allowing the joint to be cast by conventional methods with a good result. No compaction difficulties have been noticed for the CRC when casting the test specimens, but this issue needs attention when casting the joint on site. As for all steel fibre-reinforced concrete, possible surface corrosion due the steel fibres should also be mentioned.

A broad documentation of material properties such as durability, fire resistance, fatigue etc. has been supplied for CRC, [6] and [7]. Many research projects have been carried out, among others resulting in a design formula concerning the bond length for achieving full anchorage of the rebar, [5]. Thus, fatigue for bond in the form of a number of pullout tests has been investigated.

Laboratory Tests

The tests include static tests of flexural bending moments and shear forces as well as fatigue tests, conducted for a broad variation of the joint geometry in order to find the best solution. The tests have been described in [4], and only a brief summary is presented here. The first tests were executed as both shear and bending capacity tests. The initial shear tests on three specimens gave very satisfactory results, with shear capacity at failure outside the joint about 40% higher than the theoretically calculated value for the prefabricated elements; thus no further static shear tests were conducted. However, the results from the first bending tests on three specimens were not acceptable since anchorage failures in terms of pullouts were obtained before the ultimate strength of the rebar was reached, one reason for this being that not enough CRC cover was provided. Improvements of joint geometry in order to increase the bending capacity and avoid pullouts of rebar were selected as the main subject for further investigations, together with studies of the influence on the joint capacity from small variations in joint geometry.

Static Tests

The new tests therefore aimed at optimising the joint geometry with respect to flexural bending capacity, emphasizing two crucial factors for avoiding pullouts. These were sufficient concrete cover of CRC, and transverse bars simply placed on top of bottom rebars to counteract effects of tensile splitting due to bond action and to even the stresses over the joint in the longitudinal direction, since the failure mode in previous tests were rebars with a cone of CRC being pulled out. The set-up of the 4-point bending tests and the loading arrangement is shown in *Fig. 3*.

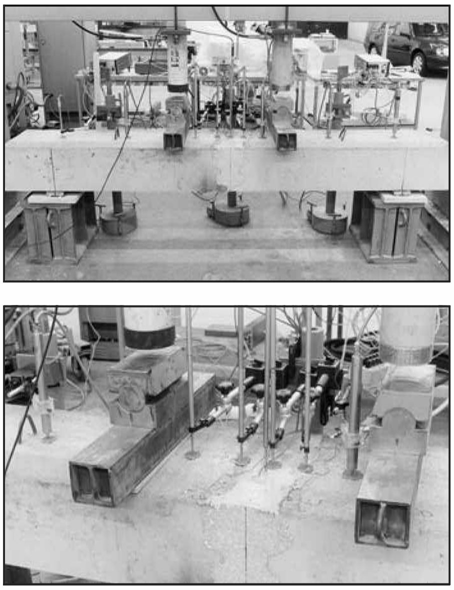


Fig. 3: Above is the set-up for static loading test. Loading consists of two symmetric point loads. Specimens span 2000 mm and the space between loading points is 600 mm. Below is a close view of the set-up. Vertical deflection was gauged by displacement transducers and strain gauges were attached to the reinforcing bars to be spliced, one at each side of the interface between the CRC and the ordinary concrete. Essentially the same set-up is used for two of the series in the fatigue test

The static tests were carried out on beams similar to actual traffic slabs, consisting of 10 specimens and 5 different configurations. The compressive strength of concrete at 28 days was 57 MPa measured on cylinders (Ø150 ξ 300 mm^2, cured in water), while the yield stress of the 16 mm reinforcing bars was 564 MPa, failing at approximately 660 MPa. The compressive strength of the CRC at 28 days was 150 MPa measured on cylinders (Ø100 ξ 200 mm^2, cured at room temperature).

Splicing of the bottom reinforcement was the most critical part, since the tests were done with a positive flexural moment. While the top reinforcement is better embedded in CRC, this would be a more favourable situation had the loading been reversed. The height of the tested specimens was 260 mm, the width was 440 mm, and two 16 mm bottom rebars (the nominal cover of ordinary concrete was 51 mm in the precast elements) were spliced in the joint. The results and a specification of the tested specimens are shown in *Table 1*.

All bending failures took place outside the connection and there were no visible cracks within the CRC in the joint. All failures occurred after yielding of the reinforcement, and most specimens achieved an ordinary bending failure with crushing of the concrete in the compression zone. The lowest failure loads were achieved for the two specimens without transverse reinforcement in the joint; these failures, being the results of rebar pullouts, led to a more brittle behaviour. One of the two specimens with reduced lap length and width of the joint exhibited a lower failure load, although the two specimens failed in different modes (compare *Table 1*). The conclusion is that the presence of a transverse bar is crucial for ductile behaviour, and that 80 mm is close to the critical lap length – but also that the joint is very sensitive to variations in the geometry parameters, as is obvious from the small dimensions of the joint.

Fatigue Tests

Yet another series of tests was carried out to investigate the performance under fatigue, which is essential to establish proper documentation in order to receive authority approval. The loading was based on the Swedish design code for bridges, in order to verify that the specimens could withstand at least 400 000 load-cycles with the maximum allowable reinforcement stress range, since this is the design criterion for similar slabs cast in situ. The maximum stress range allowed for reinforcement is given in the Swedish concrete design code as 216 MPa / γ_n for 400 000 cycles. The partial safety factor γ_n is set to 1.2 for safety class 3, which is applicable in Swedish bridge design.

The optimised joint geometry inferred from the static test was used in these test series, featuring a lap length and width of joint of 100 mm, two 8 mm transverse rebars in the joint, and 20 mm concrete cover of CRC. Six specimens with three different configurations and two different set-ups were tested. Two of the test series, UP1 with medium content of reinforcement (two Ø16 mm bottom rebars) and UP2 with high reinforcement content (three Ø16 mm bottom rebars), were performed on similar specimens (two specimens for each series) as in the static tests (compare *Fig. 3*), the goal being to exhibit sufficient fatigue strength in pure bending to withstand the conditions described above with some margin. A maximum stress believed to be substantially higher than the actual maximum stress levels for fatigue in

Specimen No.	Width of joint (mm)	Cover of CRC (mm)	Trans-Bars	Crack load (kN)	Yield load (kN)	Maximum load (kN)	Deflection at max load (mm)	Type of failure
1	100	20	Ø8	19,2	59,5	79,8	47,7	B
2	100	20	Ø10	19,9	61,8	84,9	47,9	B
3	80	20	Ø8	18,4	61,8	81,6	41,3	B*/
4	100	15	Ø8	20,0	60,0	82,9	49,1	B
5	100	15	Ø8	19,3	61,3	83,9	45,0	B
6	80	20	Ø8	13,5	63,8	78,6	30,5	A
7	100	20	None	18,7	61,9	77,9	30,7	A
8	100	20	None	18,0	61,6	76,7	29,0	A
9	100	20	Ø8	17,9	59,8	82,9	51,6	B
10	100	20	Ø10	18,1	61,0	86,9	51,6	B

Notes: Lap length is the same as the width of joint. The given load is one of the point loads (the values must be doubled for the total load). Bending failure is specified by index B (/ means a less ductile behaviour in the post-peak range due to insufficient bond), and anchorage failure (pullout) is specified by Index A*

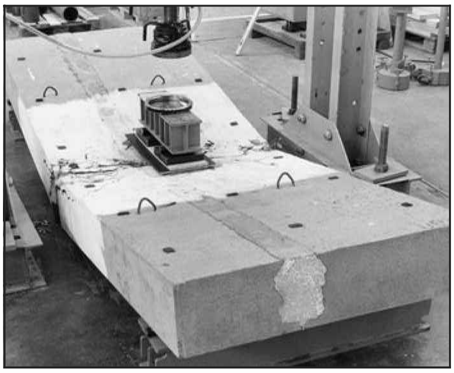
Table 1: Results from the static tests

bridge slabs was chosen, about 300 MPa, as the maximum load was set to give a flexural bending moment somewhere close to 70% of the ultimate design strength in bending. The minimum load was then calculated to achieve a theoretical stress range of about 1.15 times the nominal value from the code, resulting in 250 MPa stress range in the rebars.

The two specimens in series UP3 were oriented in the transverse direction compared to the other specimens (i.e. in the same direction as the actual deck slab spans the distance between the bridge beams, compare *Fig. 4*), with the joint along the elements, since the primary aim was to evaluate joint influence on fatigue and capacity in the direction of the main bending moment. The height and width of these specimens were 260 mm and 1200 mm respectively. A secondary issue of interest was transfer of shear force under cyclic loading in combination with bending; hence the joint was placed in an eccentric position of the jointed specimens, in order to achieve shear load transfer in the joint. The maximum load was set to 135 kN (1.5 times the fatigue load prescribed in the bridge code) distributed on a loading area of 200×600 mm^2 representing a wheel's pressure. The distance between the supports was set to 2880 mm, chosen in combination with the minimum load to reach a reinforcement stress range approximately at the same level as for the other test series.

For all series in the fatigue tests, the specimens were to be loaded statically to failure if they sustained more than 800 000 load cycles. One specimen after the ultimate static loading to failure and the set-up of the UP3 series can be seen in *Fig. 4*.

The compressive strength for ordinary concrete at 28 days was 80 MPa measured on cubes ($100 \times 100 \times 100$ mm, cured according to Swedish standard), and when testing the last specimens approximately a month later, the compressive strength had reached 64 MPa measured on cylinders (150×300 mm^2). 16 mm reinforcing bars of the same batch as in the static tests were used, and the compressive strength of the CRC at 28 days was 200 MPa measured on cubes ($100 \times 100 \times 100$ mm^3, cured according to Swedish standard), while at the time of the last testing, the strength had increased to 214 MPa.



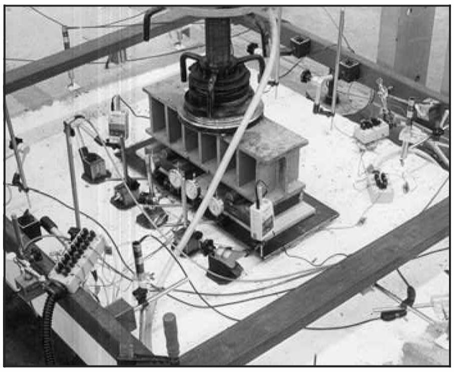


Fig. 4: Above is one of the specimens of fatigue test series UP3, after ultimate static loading to failure. Below is a close view of the set-up for the same series. The eccentric longitudinal joint and the centric loading simulating a wheel's pressure can be seen in the pictures

As the load cycles continued, the number and width of cracks increased, although no cracks were observed in the CRC joint. Results of the fatigue test series are shown in *Table 2*.

Specimen UP2:2 failed in anchorage for one corner rebar before the target of 400 000 load cycles was reached, due to insufficient CRC cover for this bar because of a misalignment and because all corner bars in this series had been placed too close to the outer side of the formwork. However, the other two bars in the specimen failed in normal fatigue bending failure without being pulled out. Specimen UP2:2 is also the only one of all specimens that failed in the interface between CRC and conventional concrete, which occurred due to the pullout of the corner bar. For all other specimens the fatigue failure occurred in the prefabricated concrete elements. For the UP3 test series it was noticed that the longitudinal joint caused a significant stiffening of the specimens.

Type of specimen	Specimen No.	Failure type	No. of load cycles	Crack load (kN)	Yield load (kN)	Failure load (kN)	Deflection at max load (mm)
UP1	UP1:1	F/B	423 000	22	–	–	–
	UP1:2	F/B	506 000	20	–	–	–
UP2	UP2:1	S/B	820 000	–	201	249	43,4
	UP2:2	F/A	382 000	30	–	–	–
UP3	UP3:1	S/B	802 000	69	318	373	49,9
	UP3:2	S/B	800 000	80	322	385	56,3

Notes: Index F/B indicates fatigue failure in bending, index F/A specifies fatigue failure in anchorage and index, and S/B indicates failure under static loading after reaching more than 800 000 load cycles (deflection and load, other than crack load, can only be shown for these specimens). The given load is the total load on the specimens.

Table 2: Results of the fatigue tests

Finite Element Analysis

Performance of the Analysis

In addition to the laboratory tests, the joints have been analysed by using the non-linear finite element programme [8]. A plane stress model was established in two dimensions based on non-linear fracture mechanics. Cracking of the concrete was modelled with the rotating crack approach based on smeared cracking and total strain. Plasticity models accounted for the non-linearity of concrete in compression as well as for the reinforcing steel. Interface elements between the concrete and reinforcement elements was used to model the bond-slip behaviour for the reinforcing bars to be spliced in the joint, while bars at the upper edge were modelled by assuming full interaction between concrete and reinforcing steel through the 'embedded' option in the FE programme. Only one half of the actual specimens were modelled, since the symmetric conditions offered this opportunity to reduce the size of the model. In return, a more dense mesh could be used. The density of the mesh was rather high in the central part of the model constituting the joint and its surroundings, whereas a significantly less dense mesh was used in the outer parts of the model closer to the supports, as can be seen in *Fig. 5*.

The non-linear analysis was performed with the material parameters achieved from the tests as presented above. However, since only the compression strength was measured for the ordinary concrete and the CRC, other material parameters were chosen according to recommendations in the literature. The fracture energy, tensile strength, and bond-slip behaviour of the reinforcement were all modelled according to Model Code 1990 [3]; for the bond-slip, 'good bond conditions' but 'unconfined concrete' were assumed. Since values recommended in literature for Young's modulus resulted in far too stiff behaviour in the analysis, a value based on empirical experience was chosen in order to enhance the performance of the analysis. In addition, a significant localisation of concrete in compression was achieved in the first analysis, resulting in a much too steep behaviour of the descending branch and a very brittle failure. To avoid this, the stress-strain relationship in the post-peak part was softened.

Furthermore, since the first crack in the test specimens occurred right outside the interface between the concrete and the CRC, the FE elements representing concrete below the reinforcement in the first row outside the interface were somewhat weakened in order to emulate the crack behaviour from the tests. Otherwise, full interaction between concrete and CRC was assumed, since there were no visible cracks inside the connection in the tests; thus no special interface elements were modelled here. Material properties for CRC, other than the measured compressive strength, were evaluated from [9] and [10]. For bond in CRC [7], an approximate bond-slip relationship was derived from results of pull-out tests. Since the numerical treatment of the bond-slip behaviour in the FE programme is unable to represent the splitting stresses actually occurring in combination with the bond stresses for ribbed bars, the bond-slip relationship in the CRC had to be slightly altered between the analyses to distinguish the difference between joints with transverse bars (counteracting the splitting tensile stresses) and those without. Moreover, the reduction of bond stress when yielding of reinforcement occurs cannot be captured by the FE analysis either, while this also justifies the alterations mentioned. Material properties used in the analysis is exhibited in *Table 3*.

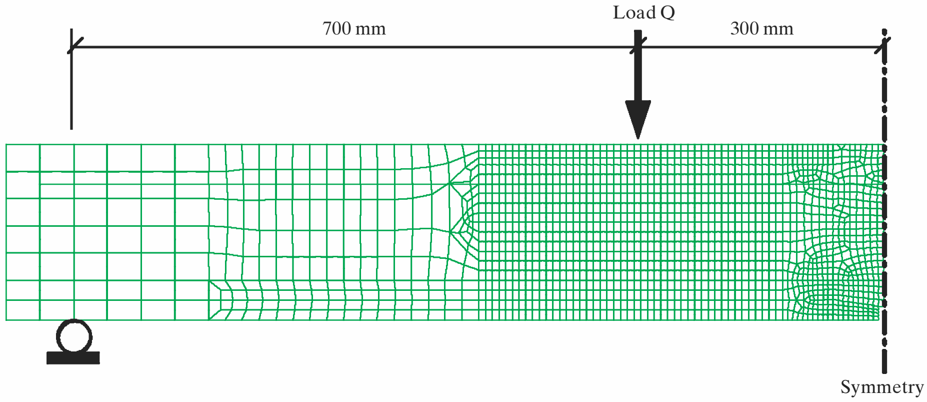


Fig. 5: The mesh of FE model

Discussion of Results

The objective of the analysis was to evaluate the mechanical behaviour of the joints through establishment of a model capturing results that coincide with the results from the tests, thus confirming the model. Some results from the analysis are shown in *Fig. 6*, i.e. the results from the static test of specimens 9 and 8 are compared to the output from the corresponding analysis for load versus mid-span deflection. Specimen 9 represents the geometry with a transverse rebar in the joint that was sequentially used in the fatigue tests. Specimen 8 has the same geometry but without the transverse reinforcing bar inside the joint.

There is a reasonable match between the curves from the FE analysis and the tests respectively. The difference in ductility between the two specimens is obvious both from the test results and from the analysis, whereas the latter shows a less ductile behaviour due to rebar being pulled out of the CRC matrix after yielding of the reinforcement, as has been mentioned.

The results from the analysis indicate that bond capacity of the joint without transverse rebar is about 75% of the capacity with the rebar present in the joint. However, the conclusion that can be drawn from the analysis is that it is somewhat difficult to model the splicing of reinforcement and the influence of the transverse rebar in a two-dimensional model, since the only parameter available to alter is the bond-slip behaviour. No means of accounting for the corresponding splitting tensile forces is possible, as mentioned. Thus it would be very interesting to conduct three-dimensional analysis using a bond model capable of describing these actions, for example the bond model described in [11]. Furthermore, if the FE model were developed and expanded into three dimensions, the results from the test could be generalised and, for example, a parametric study could be performed to further investigate the sensitivity to variations in the geometry that was detected in the laboratory tests. This would also allow some conclusions to be drawn about guidelines for design as well as for production (e.g. tolerances, etc.). Moreover, it would provide opportunities to enhance knowledge about the mechanical behaviour of the joint and especially about the influence of the transverse rebar. In this context it is important to take the rotation capacity of the joint into account, since the stiffness of the different concretes influences the ductility of the whole structure. Therefore it may be that design methods of non-linearity

Material	Type	f_{ccm} f_{sy}	f_{ctm} f_{su}	E_{cm} E_s	G_f
		(MPa)	(MPa)	(GPa)	(Nm/m^2)
Concrete	C60	56,9	4,0	30,0	101
CRC	–	149,9	10,0	48,7	12000
Reinforcement	K500	564	662	210	–

Table 3: Material properties used in the FE analysis

or simplified methods of redistribution of moments cannot be used, this is however more important to investigate for applications other than the studied.

A specific issue noticed in the fatigue tests was the divergence between the theoretically calculated minimum rebar stress and the actually measured stress in the tests, the latter being significantly higher than the former. Since such behaviour was not anticipated, it was initially somewhat worrying, as it appeared to result in a lower stress range than expected over a load cycle. A theoretical explanation was required and, on second thought, it was obvious that the phenomenon was a result of the location of the strain gauges on the rebars. The centres of the gauges were placed on the rebars at a distance of 25 mm from the interface between concrete and CRC. But the theoretical calculations are valid only in the middle of a crack, and the impossibility of ensuring placement of strain gauges in cracks led to the divergence, as discussed in the following. A background to the discussion can be found in *Fig. 7*.

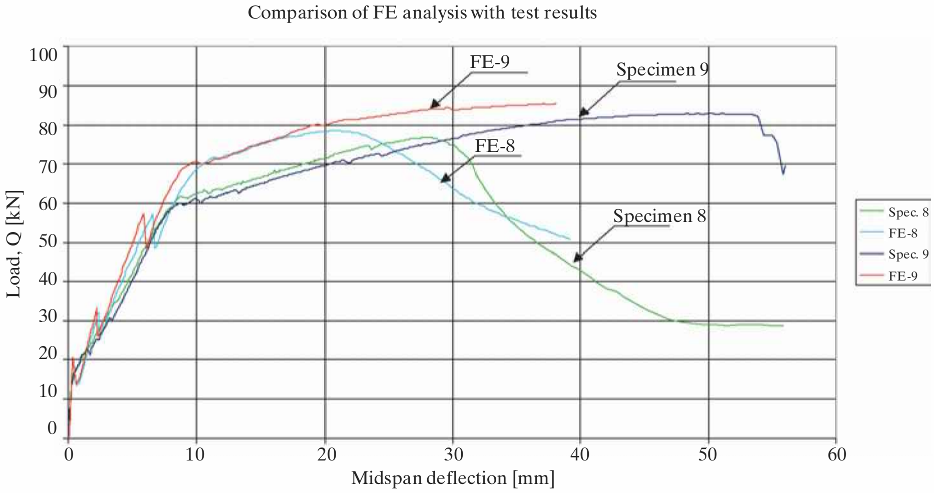


Fig. 6: Comparison of FE analysis with test results for specimens with (Specimen 9) and without (Specimen 8) transverse rebars in the joint

The reinforcement is in the elastic range during the whole procedure of the fatigue tests, and the steel strain in the cracks is many times larger than the concrete strain when in the higher range of the loading. It seems appropriate to assume that a stabilized cracking phase has superseded the initial crack formation phase (compare e.g. [12]) in this stage of the loading sequence, although the crack pattern when subject to cyclic loading cannot be considered completely stabilised because of the redistribution of bond stresses. However, due to transfer of stresses from reinforcement to concrete through the bond stresses, the actual stress and strain (at high loads) in the reinforcing steel decrease as one moves farther away from the crack surface, until the bond stresses are reduced to a minimum value close to the midpoint between two cracks.

As the specimen is unloaded, the bond stress decreases rapidly due to load action, and ultimately the sign of the bond stresses is reversed (starting at the crack and progressing towards the middle as the load is decreased) and a 'reversed friction' is activated, actually causing a compressive stress in the uncracked concrete between the open cracks. Although on a smaller scale, the situation could be compared to the case of anchorage regions at beam-ends for post-tensioned concrete. While reaching the minimum load value, or even zero load, there is likely to be a significant proportion of reversed bond stresses that are activated due to the difference in strain between reinforcement and concrete at the previous maximum load. Thus there will be tensile reinforcement stresses and compressive concrete stresses between the cracks (increasing as one moves toward the midpoint between cracks), even if the load is decreased to zero. Consequently, the cracks remain open to an extent that cannot be neglected, as was also observed in the fatigue tests. This occurs even if the reinforcement stress is zero in the crack plane in the case of zero load, as inferred from the theoretical calculations.

The relative influence of this bond-slip effect is larger in the minimum load case than in the maximum load case, due to the difference in stress levels. Hence, according to the discussion, compared to the theoretical calculations for the crack plane, there will be a somewhat lower steel strain measured near maximum load, and a significantly higher steel strain measured when the load is reaching its minimum value, if the strain gauges are placed at some distance from the crack.

To investigate this behaviour, the finite element FE model was adjusted by including additional non-linear elasticity interface elements (to be activated as soon as the relative slip changes sign when unloading) between the concrete and reinforcement elements, thus simulating the effects of the 'reversed friction' as the reinforcing bar is unloaded, and the model was analysed for some initial load cycles. The results in comparison with results from the unmodified model and measured values from one of the fatigue-tested specimen are presented in *Fig. 8*, showing results for one reinforcing element from the modified model (element 2810) further away from the initiated crack than the other element (element 3260), which is quite close to the crack, as well as results for the element in the crack from the unmodified model (element 3260). As can be seen, this could indicate a reasonable confirmation of the previous discussion, although the conformity of the curves in comparison with measured values are not very good, possibly due to the difficulties of accurate modelling of the 'reversed friction'. Further analysis with an expanded three-dimensional model would undoubtedly help to clarify the matter.

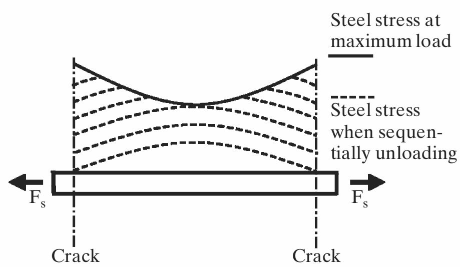


Fig. 7: Schematic presentation of the variation in tensile stress between two cracks due to bond action for a reinforcing bar in tension while unloading from maximum load. Mainly tensile loading is assumed. F_s is the tensile force in the rebar

Summary and Conclusions

A new concept for high performance connections between prefabricated concrete elements in bridges has been investigated with the goal to design a joint that makes the surrounding elements continuous, but still a very small joint that is easy and fast to produce. A special steel fibre-reinforced high performance concrete called CRC, is used in the joint. Laboratory tests as well as finite element (FE) analysis have been conducted.

When compared to the adjoining prefabricated elements, the joint will be over-strong if a proper detailing with a sufficient lap length is provided. It was concluded that a lap length of 100 mm was adequate if two transverse reinforcing bars were simply put on top of the spliced bars. Thus, continuous structural elements treated as a monolithic member in design can be created of precast concrete connected by this joint. The tests also show that, due to the small dimensions of the joint, it is rather sensitive to deficient compliance to tolerances and workmanship on site. A great deal of attention will therefore have to be paid to quality control at the construction site when using this type of connection.

For one of the specimens in the fatigue tests, UP2:2, an anchorage failure in terms of a pullout was observed. However, since the failure occurred in just one corner bar with insufficient CRC cover, while the other bars performed well, this can be disregarded if proper CRC cover is provided for the corner rebars.

Another conclusion is that three-dimensional analysis could provide additional understanding about the mechanical behaviour of the joint, to be used e.g. in guidelines for design and construction. The joint is obviously applicable in this context, and it would be elucidating to perform a full-scale pilot project as a step following eventual three-dimensional FE analysis.

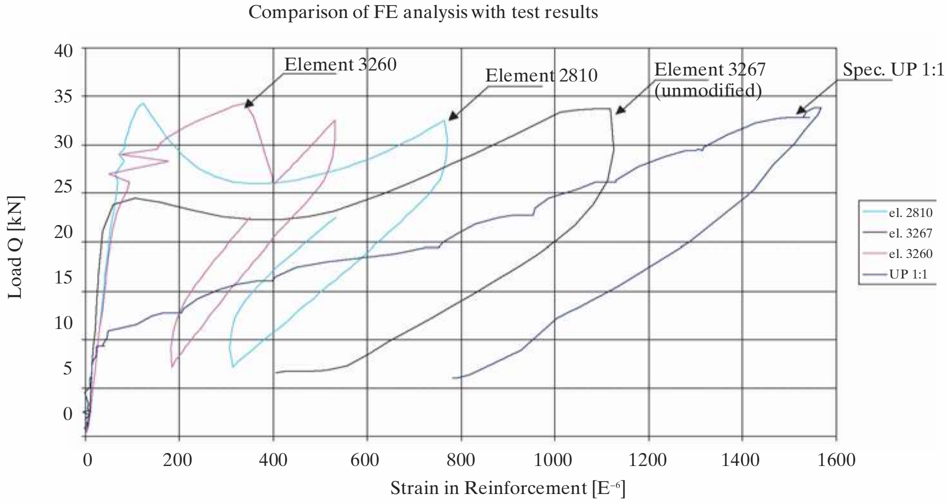


Fig. 8: Comparison of steel strains from analysis of the modified and unmodified FE models with measured values

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Paper V

The *i-bridge*, a novel bridge concept, Feasibility studies
embracing industrial bridge engineering
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The *i-bridge*, a novel bridge concept
Feasibility studies embracing industrial bridge engineering

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Summary

Feasibility studies on a novel bridge concept embracing industrial bridge engineering, conceptual design and finite element (FE) analyses have been carried out at Chalmers University of Technology. The *i-bridge* concept consists of v-shaped glass fibre reinforced polymer (GFRP) beams reinforced by carbon fibre reinforced polymer (CFRP) profiles. The deck consists of GFRP plates in composite action with ultra-high-performance steel-fibre reinforced concrete (UHPSFRC). A general description of the bridge concept as well as the conducted initial investigations and numerical analyses are presented.

The investigations performed indicate that the bridge concept could be realised from a technical structural point of view. In addition, the industrial characteristics proposed aims at ensuring efficient production and operation of the bridge.

Keywords: Bridge concept, UHPSFRC, GFRP, CFRP, Conceptual design, FE analyses.

Introduction

The feasibility study of the *i-bridge* concept is a part of a research project concerning industrial bridge engineering carried out at Chalmers University of Technology; see [1]. The aim of the study is to elucidate the possibilities offered by new technology, new materials and other advancements when developing concepts of industrial construction (i-construction), and especially industrial bridge construction concepts. Hence, it is an effort to demonstrate how new developments can be utilised in bridge construction.

The study is divided into three parts and each part is presented in an article. This article, forming the first part, is dedicated to a general description of the concept as well as the initial investigations and numerical analyses conducted. The second part; see [2], concerns an experimental study of bonded interfaces in the bridge deck and includes laboratory tests. The third and last part, in [3], deals with the assessment of a test beam and covers both laboratory load testing and finite element analyses.

Background

It was decided in an early stage of the research project that a feasibility study of a potentially successful industrial bridge concept was to be undertaken, including a conceptual design phase. Hence, structural engineering and design aspects were focused upon, along with industrial matters and other critical issues as stated in the following sections. The feasibility study constitutes an attempt to show how structural engineering can contribute to enhance the effectiveness of the overall construction process and encourage development. The study forms a branch in the *product development* sub-process of the industrial construction process (compare Figure 1). Still, the contribution to the overall process can be significant even though this is not the focus of the study. Thus, the main focus of the study lies in the technical domain of the process, dealing mainly with technical assessment of structural elements.

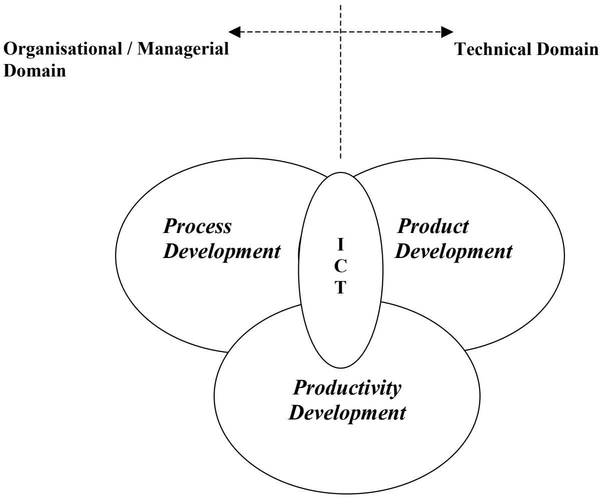

Fig. 1: The cornerstones of Industrial Bridge Construction, the three P's, from [1].

Prototype bridges

Feasibility of bridge prototypes

There is a variety of demands and requirements to consider when developing a novel bridge concept. In addition, there are many desired kinds of performance with added value that would be beneficial to achieve.

It is the aim of the feasibility study to illuminate all crucial concerns that could obstruct the concept from being realised. Reasonable validation of these matters must be provided to confirm the feasibility of the concept. A wide range of questions need to be answered. Subjects range from production matters, assembly, erection, industrial matters, economics, structural aspects, design, aesthetics, environmental issues and durability to time of construction and efficiency. To generalise, the issues are divided into two parts. The first general part mainly concerns industrial aspects and the second part concerns structural matters and design.

Materials and bridge types

To validate the structural performance in a feasibility study, material parameters as well as design requirements are needed. In addition, some crucial details must be investigated, but also the whole structure or large parts thereof. In feasibility studies, however, the areas of interest must be limited to the most essential issues. It is important to remember, though, that the investigations which are conducted in a feasibility study are initial ones, and that refinement is of course necessary in a later stage.

There are many creative developments in materials that have not become commonly utilised in construction. One group of materials still awaiting their large-scale breakthrough in construction is fibre reinforced polymers (FRP). Great effort has been put into research and development of these materials in order to enhance their utilisation in construction. Until today, however, they are mostly used in special applications and not generally widespread, except for strengthening of structures where the use of FRP is the state-of-the-art. Design guidelines for strengthening of concrete structures can be found in e.g. [4]. The large advantages of FRP, as stated in [5], are their excellent specific strength and that they can easily be formed into any shape, are largely corrosion-free and are largely resistant to fatigue. Structural components of FRP can be industrially produced and can be erected on the construction site in a very short time without the need of heavy lifting equipment. Troyano [6] argues that the construction business is very sensitive to costs, and this is why composite materials still are a long way from becoming a building material able to compete with concrete or steel, despite their significantly higher specific strength; however, the question is not whether, but when, these materials will predominate in bridge construction. Additionally, research into new bridge concepts utilising FRP, where one of the essential issues is bonding technology, is highlighted among the most important research areas; see [5].

Another material that has been promising for quite a while is ultra-high-performance steel-fibre reinforced concrete (UHPSFRC). This is a material that has been commercially available for more than a decade, but still has not been utilised on a large scale and is currently used mostly in special applications. It is able to cope with large compression and to work in composite action with e.g. FRP. The tension capacity is preferably large enough to avoid the use of reinforcing bars. Additionally, a robust, dense and rough material is needed for the wearing surface of the deck, if the deck is to be durable even without protection from isolation. A review of different types of UHPSFRC can be found in [7].

There are numerous cases of inventive bridge designs; see e.g.[8], [9], [10], [11], [12] and [13] for some examples. Reviews of bridges containing FRP can be found in e.g. [5] and [14].

The *i-bridge* concept should of course allow efficient industrial production of the bridge with large flexibility and variation, to be able to adapt to different situations and locations. The general idea is to combine and utilise appropriate materials and to exploit their entire capacity. Hence, the bridge type and the geometry should be appropriately chosen in order to fully utilise the performance of the materials. The concept in the feasibility study refers to road bridges of normal span length.

Concerning analysis and design of bridges, a general and comprehensive review of structural matters, design requirements and different codes is given in [15].

Since both FRP and ultra-high-performance steel-fibre reinforced concrete are materials enabling a choice of tailored material parameters, i.e. engineered material, the main task for the designer is to specify the parameters to suit the design – rather than adapting a design to meet predetermined material performance, which is the common approach to design in construction.

Prerequisites and preferences for the bridge concept

The aim of industrial construction, and indeed for any industrial process, is to make products at a lower cost or alternatively to make products of higher quality at the same cost. The optimum is of course for both criteria to be fulfilled at the same time, i.e. make products of higher quality at lower cost.

Certain requirements have been stipulated, such as that the concept must fit into an industrial process and that industrial methods and developments were to be utilised. It was also an aim to try to utilise new, or in construction not commonly used materials and developments. Hence, there was a risk that the concept would not be economically competitive in the current market, but this was to be disregarded. If the concept does not seem to be feasible today for economic reasons, it should still possess a large future potential.

A summary of the desired requirements for the bridge concept is presented in Table 1.

Table 1. Desired requirements for the bridge concept.

Basic Requirements

- functionality, safety, serviceability and comfort criteria
- manufacturing suited for an industrial construction process
- sustainability, durability, maintenance and repair aspects taken into account
- quality aspects, lifetime assessment and environmental concern.
- economic, flexible and efficient construction
- short time of construction (on site)
- labour-friendly working environment

Requirements causing enhanced customer value

- substantial reduction of waste, reducing overall construction time and cost
- simple and rational construction or manufacturing, resulting in lowered construction cost
- better use of resources from a public economic point of view
- minimisation of life cycle cost and environmental effects
- improved quality aiming at zero mistakes and consequently no guarantee work
- continuous development and improvement of processes, products and productivity
- flexible process, both during construction and afterwards without disturbance due to construction work
- developed ICT systems to govern the overall process, resulting in e.g. better planning, easier procurement and less administration.
- optimisation, easy, fast and straightforward design, integrated in the overall process
- enhanced durability and reduced need of maintenance and repair; if still needed, it is to be pre-planned, easy and fast to execute
- simple and minimised work on site, and all work carried out in sequence, thus avoiding later rework
- simple and light elements to minimise transportation, avoid heavy lifts etc. and to provide efficient assembly.
- a high degree of prefabrication, preferably with no on-site work except erection and assembly
- enhanced flexibility with the possibility to adopt to different road designs
- possibility to alter the aesthetics and the appearance of the bridge
- possibility to complete the bridge in one week on site with only short interruptions in traffic (when built over existing roads)
- non-labour-intensive production and erection

Special requirement for the study

- to utilise and demonstrate new or not often used technical developments

The choice of materials for the concept fell upon FRP in composite action with ultra-high-performance steel-fibre reinforced concrete, as been indicated above. Although not always with the type of concrete chosen here, this interesting combination has been tested in several research projects before; compare e.g. [16]. The materials are glass fibre reinforced polymers (GFRP), carbon fibre reinforced polymer (CFRP) and CRC concrete, where the FRP is used in the major load-bearing components and the concrete is utilised in the deck. Among the main reasons for this choice were durability and life-cycle concerns, since there was a wish to exclude the use of reinforcing steel and to utilise more maintenance-free materials. It also seemed challenging to provide for composite action between the different materials, similar to the research done to provide for composite action between steel plates and concrete; see e.g. [17].

For the GFRP, E-glass fibres in a vinylester resin were chosen. The CFRP consists of carbon fibres of intermediate modulus (IM) in an epoxy matrix.

Compact Reinforced Composite (CRC) is a very dense, silica-fume-based ultra-high-performance concrete; see e.g. [18] and [19]. It is characterised by densely packed fine and ultra-fine particles combined with steel fibers, typically ranging from about 2 to 6% by volume. In this case 6% or about 450 kg of steel fibers per m^3 of concrete was used. The water/binder ratio is only about 0.16 and the silica fume content is 20–25 %. Special aggregates consisting of quartz sand particles with diameters up to 4 mm are used. The compressive strength of CRC is very high, normally about 150 MPa, but higher values can be reached. CRC provides exceptional bond properties and good adhesion to other materials. Furthermore, the dry mortar ready-mix used allows uncomplicated mixing and casting of the concrete; thus conventional techniques can be used.

The bridge type is a normal road bridge, while the shape of the load-bearing components was received in the conceptual design on the basis of the most important issues focused upon, i.e. efficiency and industrial matters.

Limitations

Obviously, it is not possible to cover all part of the bridge development and design in detail. Since the aim of the feasibility study is to determine whether the bridge concept is possible from industrial and structural engineering points of view, the main focus in the study lies on the industrial issues and the conceptual design phase, e.g. to ensure that the overall static system is working properly.

Thus, there are many details that are left out or not covered in depth. However, there has been an intention to adopt a holistic view in the study. Efforts have been made at least to mention all relevant issues and try to present plausible solutions, although every investigations needed to ensure their correctness are lacking.

Aspects covered more in depth in the study are the conceptual and preliminary structural design of the main load-bearing parts of the bridge, i.e. the bridge beams and the deck of the superstructure. No optimisation of the structure has been done, though. In addition, only one-span bridges were considered at this stage.

The *i-bridge* concept

Outline of the bridge concept

The conceptual design phase of the feasibility study, which is presented more thoroughly in the next section, was initiated by an estimative preliminary design carried out to outline the concept. This process ended with several possible alternatives. The four most interesting alternatives were examined in a somewhat subjective way by judging them out of about thirty different aspects. The aspects varied from economics, manufacturing, aesthetics, effectiveness, time to complete on site, erection methods and industrial potential, to purely technical matters. For each aspect, each alternative was rewarded a point which was weighted in terms of the severity of the aspect. The alternative with the highest weighted grade was chosen.

Obviously, due to the nature of a feasibility study the bridge only exists in theory as yet, and what is said about the concept at this stage consists of well-grounded but fictitious assumptions. In the following, some basic assumptions about the performance of the bridge concept are presented.

The superstructure of the *i-bridge* concept consists of v-shaped GFRP beams reinforced by CFRP profiles, with a deck consisting of GFRP plates in composite action with UHPSFRC. The CFRP profile is placed inside the beams, mainly for aesthetic and protective reasons. The concept is outlined in Figure 2–4. The bridge in the study is freely supported, while multi-span continuous bridges are excluded at this stage. The span width designed for in the study is 25 m, which is a common intermediate span that can accommodate a magnitude of bridging situations.

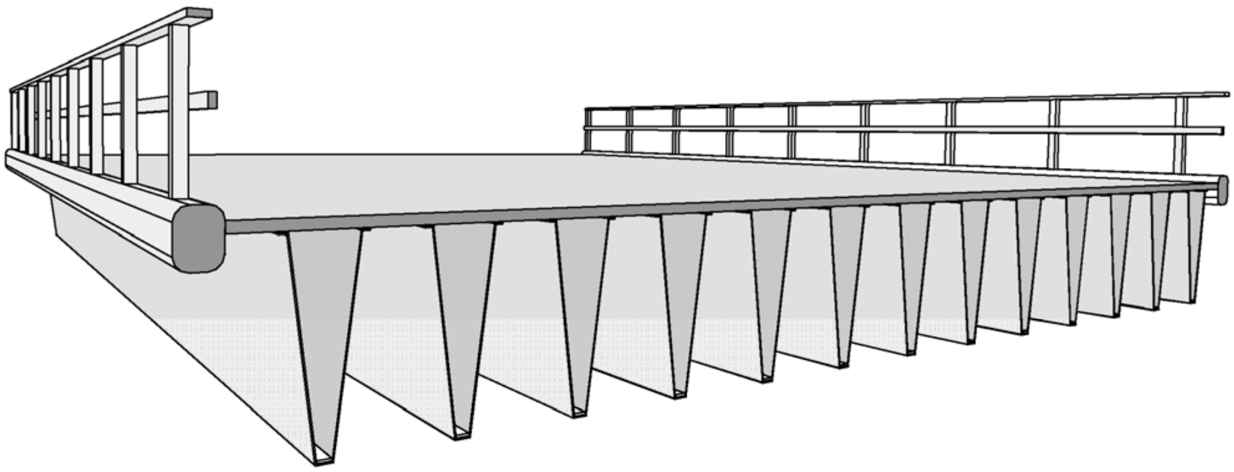

Fig. 2: Outline of the i-bridge concept, a perspective sketch illustrating the major components of the bridge.

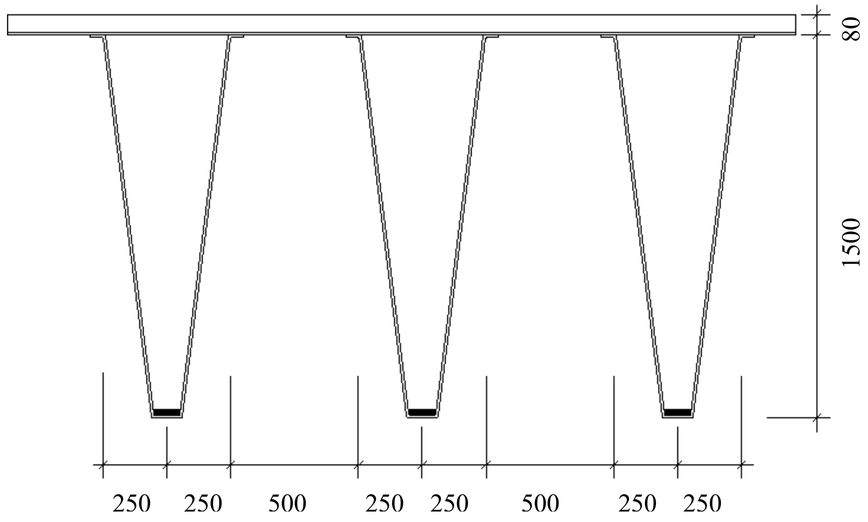

Fig. 3: Major dimensions of the cross section (in mm).

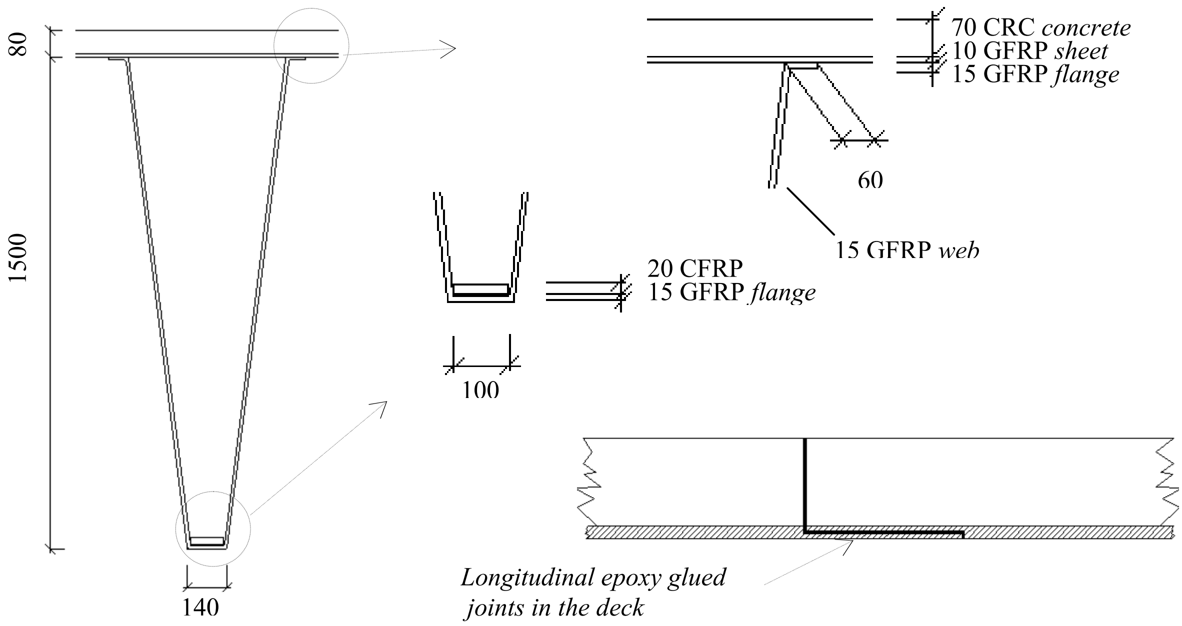

Fig. 4: Detailed dimensions (in mm).

The groundwork needed can be rapidly performed. It is envisaged that substantial savings can be achieved in the foundation and substructure due to the reduction of reaction forces as a result of the low dead weight of the bridge, e.g. the foundation can be done by means of horizontally reinforced earth. This is a very effective foundation, since the rather concentrated ground pressure from the bottom slab is rapidly distributed over larger areas in the reinforced earth; compare e.g. [20]. Hence, generally it will not be necessary to strengthen the earth vertically at all. Otherwise the lime column method will most often be sufficient in the case of worse soil conditions. Conventional piling is only needed for very bad soil conditions, e.g. for soft clay. The reinforced earth can be complemented with a lining depending on the prerequisites in each location.

The suggested substructure simply consists of a slab on top of the reinforced earth. In special cases of very good ground conditions, e.g. stiff frictional soil or rock, the reinforced earth can be excluded. However, in most cases it is needed, also for geometrical reasons in order to provide the appropriate level for the base slab without the need of abutment walls. In this way the dimensions of wing walls and end walls are minimised so that they can be prefabricated and mounted on site. Alternatively, they can be included in the substructure, integrated as a small seating with connecting wing walls. Where needed, a run-on slab secures the elimination of differential settlements. Furthermore, if reinforced earth is employed also behind the bridge, wing walls and end walls can be excluded. No detailed solutions for the foundation and substructure have been studied, but the suggested general principles are shown in Figure 5.

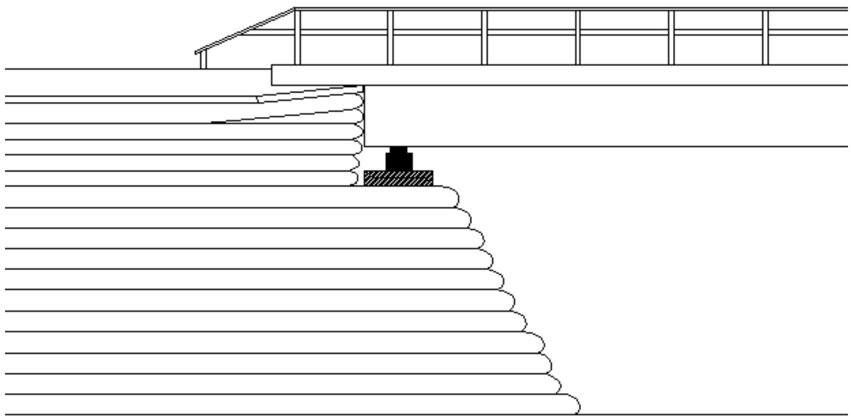

Fig. 5: An elevation showing principles for the proposed foundation and substructure of the bridge.

In the case of longer bridges with several spans, the continuity over intermediate supports can be dealt with by means of special reinforcements in the deck and one or two splices in the FRP beams. The nature and type of these reinforcements and splices must be evaluated and designed, but since only one-span bridges were currently included this is beyond the scope of the present study.

Ideas for columns in intermediate supports could be CRC-filled tubes of GFRP, possibly with a hollow section and with an inner skin also from GFRP.

There are huge possibilities to include different kinds of monitoring equipment when manufacturing the different parts, e.g. fibre optic sensors (FOS). Hence, the bridge performance along with other parameters, e.g. traffic loading, axle loads etc., can be remotely monitored by the operators. In addition, transmitters can be included in the structure to create a “smart structure”, e.g. to facilitate the assembly and erection.

Production and assembly

In the following, suggestions about the production and assembly of the bridge are presented.

All parts of the bridge are industrially manufactured indoors and subsequently assembled and erected on site. The GFRP beams and the CFRP profiles are manufactured through the pultrusion process. After the beams have been cut into the appropriate lengths, the CFRP profile is glued to the beam with an epoxy adhesive. There is also a possibility to prestress the CFRP before attaching it to the beam, thus introducing a prestressing force to the beams before the mounting of the deck. In this way it will not be necessary to pre-camber the beams. Prestressing also acts beneficially with respect to the long-term properties of the beam, and counteracts creep.

The deck is prefabricated in a factory where the concrete is cast upon the GFRP plate, which is produced by the vacuum bag process. The plate is manufactured with a special rough surface which provides a certain amount of shear resistance when the concrete is cast upon it. In general this will be sufficient to allow for the composite action in the deck. In special areas, however, where this may not be enough, there is also the possibility to increase the shear resistance through special surface treatment or through mechanical devices applied to the surface.

The deck is prefabricated in approximately 3 m wide elements with the same length as the full width of the bridge. It is jointed to the upper flanges of the beams by epoxy adhesive.

Crossbeams are installed at the supports to stiffen the beams for the concentrated reactions. They are possibly cast of CRC within the beams, and for transport reasons they are complemented on site.

The edge beams are designed to distribute the collision force from the parapet into the deck. The edge beam is made of GFRP and concrete in composite action, and it can be prefabricated as a full-length element or it can be included in the deck elements. No detailed solutions have been studied for the parapet and the edge beam, however.

The beams are installed with a certain level difference in order to accommodate an inclination of the bridge deck for the water to run off. In case of double-sided inclination, the deck elements are produced with a small bend.

The assembly is characterised by a stepwise erection in just a few phases. When the foundation and substructure are completed, the beams are placed on rubber bearings on the seatings by a mobile crane. The beams do not weigh more than about 3 tons, so a normal crane will be sufficient. If the conditions are favourable, the erection can even be done directly from trucks equipped with cranes. After the placement of temporary bracings, the deck is put in place after the epoxy has been spread on the upper beam flanges, one after the other. The weight of the deck elements are also reasonable, depending on the width of the bridge they weigh up to about 6 tons. In the same phase the deck elements are jointed by means of epoxy. Work with epoxy is carried out of specially trained personnel under special safety precautions.

Industrial characteristics

Here a vision is presented of how the industrial features and the construction process for the *i-bridge* concept could look.

The most obvious industrial feature is the production. All parts of the bridge are manufactured indoors in factories and transported to the site for assembly and erection. All production in the factories can be automated to a large extent. The factories manufacture on demand and the elements are produced according to the just-in-time philosophy, eliminating the need to keep components in stock. Lean production is adopted and thus the flows through the factories are facilitated in order to eliminate congestion and reduce storage.

All elements are light and easy to transport. The elements are densely loaded in order to minimise the transports, e.g. the beams are simply stacked onto each other. Hence, a few trucks are generally enough to transport a normal-sized bridge to the site of erection.

Besides the substructure, the on-site work is reduced to erection and assembly of the elements, which is rapidly performed. As no part is particularly heavy, a small or normal-sized crane is sufficient to perform all lifts for completing the assembly, as been mentioned. The bridge is ready for use a few days after erection.

It is envisaged that massive support from information and communication technologies (ICT) is utilised throughout the manufacturing process. A database governs the total process as well as all the sub-processes. The erection is supervised by GPS via cast-in computer chips, ensuring correct positioning of each element. Design and construction are fully integrated. All bridges are pre-built in the computer in order to check procedures and correct mistakes, i.e. computer-integrated construction (CIC) is used. The bridge is designed from data bases where data are reused and only the project unique parameters need to be put in. Hence, the role of the designer is more one of product development, refinement and optimisation than the common design from scratch which is usual in construction. The ICT systems are also used for production planning, as a management tool, for risk assessment, for quality control, as supply network tool, in the electronic trade, for continuous improvement, for educating the staff and for feedback.

There is a huge flexibility of the concept and the production can easily be altered to adjust to specific situations. It is very easy to change parameters in production, material properties etc. Hence, it is very easy to change the span and the cross-section of the bridge, and all parts are adjusted accordingly. Since concurrent engineering is adopted, this is also easily supported in design where material requirements are altered to the specific case before being sent to the shop for fabrication, resulting in high material utilisation and a significant reduction of waste. Hence, engineered materials in the fullest sense are used.

Other aspects

There are several aesthetic programs for the customer to choose from, each designed to meet different situations. For example, different colours, screen print, alteration of the parapets, and different linings such as stainless steel, glass, etc. are provided. In addition, there are possibilities for the customer to make own one-of-a-kind demands.

The environmental effects are minimised throughout the whole process. All aspects are covered in each step, from production and erection through operation and to final demolition. Such things as minimising energy consumption, use of environmentally friendly materials, no discharge of possible environmentally hostile materials, and minimising pollution to water and air are focused on continuously.

The durable and sustainable materials ensure a low life cycle cost and minimise the need of maintenance and repair. Each bridge is produced for a specific lifetime over which it is maintenance-free. The only time when maintenance and repair are needed is in the case of an accident that causes deficiencies. After the end of the lifetime, the bridge is demolished and the materials are reused in other applications. Only a small part needs to be finally disposed of.

It is difficult to estimate the economic potential of the bridge concept, since it contains several features which have not been used in production before. For example, the size of the pultruded beams is larger than similar current products. On the other hand, pultrusion is one of the most cost-effective ways of producing FRP products, while there is a need for large series to reach this low price per unit. The amount of CFRP is also significant, while CFRP is a material which is currently rather expensive. Hence, an economic estimation reveals that the initial cost today would be more than double compared to a conventional cast-in-place concrete bridge. In best case, if savings in groundwork and substructure as well as the effects of large-scale production and possible decreasing price of carbon fibres are taken into account, the bridge would cost about the same as the cast-in-place alternative. However, in a life cycle design perspective (compare e.g. [21]) the prospective for the bridge concept looks much better due to its anticipated low life-cycle cost. In addition, the short construction time, especially on site, and the improved working conditions for labourers will add further to the benefits of the concept.

Conceptual design and FE Analyses

Basic conditions

Apart from the evaluation of alternatives mentioned in the foregoing, the conceptual design phase of the feasibility study consists of the following parts. The first part concerns the identification of the critical issues and a preliminary evaluation of their consequences and possible solutions. The next part covers studies more in detail of problems that could not be solved in the previous part. The final part is FE analyses to validate the concept. These parts are summarised in this chapter.

Additionally, as mentioned earlier, an experimental study of bond in the material interfaces and the assessment of a test beam covering both laboratory load testing and finite element analysis have been undertaken. These parts are presented in separate articles. Many structural issues encountered in the study are somewhat similar to those experienced in [10].

One obstacle when designing FRP components is the lack of generally accepted design codes and guidelines. There are, however, some examples as summarised in e.g. [5]. The code chosen is the quite comprehensive Eurocomp [22] which, despite the fact that it is mainly valid for GFRP, is also adopted for CFRP.

A review of bridge codes can be found in e.g.[15]. The code used in the study is the Swedish Bro 2004 [23], which forerunner was used in design of FRP in [24] and which is believed to cover most design situations. It is mainly the traffic loads, the load combinations and deflection requirements etc. that have been taken from the bridge code. For concrete in general, Modelcode 90 is adopted; see [25]. In addition, although not used in this study, the Canadian bridge code provides for designs of FRP bridge structures, see [26].

The GFRP consists of E-glass in vinyl ester matrix. The material properties for the GFRP were estimated by assuming a fibre volume fraction of 60% which is a normal level to be reached in pultrusion. All essentially bi-directional laminates were assumed, with only a small portion of the fibres (about 20%) in the diagonal directions.

The CFRP is made of unidirectional IM (intermediate-modulus) PAN carbon fibres embedded in epoxy resin. The volume fraction of fibres is assumed to be 60%.

Materials data for the joint epoxy were received from [27] and from the manufacturer. In addition, evaluation of the bond tests, reported in [2], gave supplementary information.

Input data for the CRC were fetched from the materials tests as well as from [18].

For the interfaces between CRC and GFRP, parameters were evaluated from information about the response in shear and tension from the bond tests.

The most essential assumed material properties and the partial safety factors used in design and FE analyses are presented in Table 2.

Table 2. Some material properties and partial safety factors used in design and FE analyses.

CFRP	**GFRP**	**Partial safety factors, FRP**	**CRC**	**Partial safety factors, CRC**
E_{xk} = 160 GPa E_{yk} = 8 GPa E_{zk} = 8 GPa σ_{xtk} = 2000 MPa σ_{xck}= -1500MPa σ_{ytk} = 45 MPa σ_{yck} = -160 MPa σ_{ztk} = 45 MPa σ_{zck} = -160 MPa	E_{xk} = 21 GPa E_{yk} = 21 GPa E_{zk} = 8 GPa σ_{xtk} = 340 MPa σ_{xck} = -160 MPa σ_{ytk} = 340 MPa σ_{yck} = -160 MPa σ_{ztk} = 8 MPa σ_{zck} = -45 MPa	*<u>ULS</u>* γ_m = 1,5 (strength) γ_m = 1,1 (stiffness) *<u>SLS</u>* γ_m = 1,3 (strength) γ_m = 1,1 (stiffness)	E_{ck} = 46 GPa f_{cck} = 125MPa f_{ctk} = 14 MPa $f_{ctk,cr}$ = 6 MPa	*<u>ULS</u>* γ_m = 1,8 (strength) γ_m = 1,44 (stiffn.) *<u>SLS</u>* γ_m = 1,0

Critical issues and suggested solutions

The conceptual design phase started with estimative calculations to capture the gross dimensions of the structural elements. Once the evaluation of the different alternatives and the subsequent choice were made, the calculations were somewhat refined before entering the finite element (FE) analyses.

The preliminary design in the serviceability limit state (SLS) and the ultimate limit state (ULS) were verified in the FE analyses. Some critical design issues are discussed in the following.

Serviceability limit state – SLS

The issue governing the design is the deflection criteria in SLS due to the rather low elastic modulus of the GFRP. Hence, this was evaluated as a starting point, while all other requirements were checked subsequently.

It seems a borderline case whether the deck is cracked or not in the transversal direction in SLS. However, should it be necessary to evaluate the crack widths for long-term loads, it is assumed that this will not constitute a problem since maximum crack width at peak load in failure is about 0.2 mm according to [18]. Hence, the crack widths for long-term load will not influence the durability; see [28].

Other important issues checked are the dynamic response, creep under sustained loading conditions, and shrinkage in the concrete.

Ultimate limit state – ULS

Besides the global stress level considerations in ULS, local problems have been estimated and checked according to Eurocomp, e.g. stability problems, compare e.g. [29] regarding buckling of FRP.

Fatigue

Fatigue problems are mostly related to the GFRP, since neither CRC (in compression) nor CFRP is very sensitive to fatigue. Fatigue of FRP has been investigated e.g. in [30]. Estimations suggest that there seem to be no fatigue problem for the structural components, but testing is proposed since the number of stress cycles for design of bridges is more than recommended for GFRP in Eurocomp. Additionally, for the interfaces it is also a bit difficult to judge. Hence, appropriate tests need to be carried out in order to verify the fatigue performance for the whole structure.

Ultimate failure

The ultimate failure is likely to be brittle due to the elastic behaviour of the FRP. It is expected, however, that an early warning and a somewhat semi-ductile response are achieved by local loss of bond in the interface between concrete and GFRP in the vicinity of the load, followed by excessive deflections. This behaviour was to be evaluated in the experiment with the prototype test beam, but could unfortunately not be confirmed due to an unexpected delamination failure in the GFRP deck plate, compare [3].

In addition, it could be possible to use fibre optic sensors (FOS) as an early-warning-of-failure system.

Assembly

The assembly load cases have not been considered critical at this stage, but need to be investigated in the subsequent phases.

Accidental load and sabotage

The sabotage issue must be treated on a case-to-case basis, with relevant risk assessment for each location. Some geometrical design measures can be taken, e.g. to make the assessment under the bridge difficult by means of a steep slope, etc. When there is a high risk of sabotage, one solution could be to incorporate a thin uncuttable layer in the underside of the beams to protect the CFRP.

Regarding fire, a comprehensive risk evaluation must be carried out for the specific location of each bridge. Based on the risk assessment, appropriate measures can be taken. In general, FRP do not withstand fire very well. It is mainly the matrix that is susceptible and degrades first at high temperatures. The glass transition point Tg (the point where polymers rapidly lose their strength) is about 100–150 degrees Celsius for the normal resins used, and even lower for adhesives. Although more expensive, the phenolic resins represent much better fire performance with Tg = 250 degrees Celsius, low flammability, low spread of flame and little smoke.

The heat transition for FRP, however, is about 200 times lower compared with steel. Hence, a fire design based on the risk assessment must be done, ensuring that the evacuation time for people nearby is sufficient in case of an emergency. Solutions can be physical protection of the structural components or the use of phenolic resins in the FRP.

Collisions must be taken into account in the design. In general, the best solution is to prevent accidents from happening by the geometrical design, etc. In the cases where this is not possible it must be validated that the superstructure will withstand the loss of one or several of the beams. Collision load on the parapets must of course also be taken into account.

GFRP-related issues

FRP exhibit creep, i.e. time-dependent deformation under constant load. It is mostly GFRP that are susceptible to creep, while for CFRP this is not so pronounced. For high stress levels or elevated temperatures under long-term loads, the creep can develop into stress rupture. A rough estimation for the bridge concept shows that the low long-term load levels (due to the low dead weight) is likely to give no problems with excessive creep, but the subject needs to be looked into more thoroughly. It is most probable that it is the creep and the physical ageing of the GFRP that determines the design life of the bridge. In addition, stress relaxation is a phenomenon that can be present at high stress levels, but this is not the case here.

GFRP in general can suffer from alkaline attacks. Hence, special precautions need to be taken in case of concrete exposure as in the bridge deck. The vinyl ester resin and the E-glass should provide assurance in this respect. In addition, if the fibres are attacked by the environment stress corrosion might occur, but this is mostly in acid environments, see e.g. [31].

The issue of hygrothermal stress, i.e. internal stress as a result of variations in temperature or humidity, is counteracted by the use of symmetric and balanced laminates.

Detailing

The detailing has been covered by means of general solutions, but no detailed design has been carried out; this will have to be done in the continued work.

Prestressing of CFRP

The possibilities to prestress the GFRP beam, by means of prestressing the CFRP profile before attaching it to the beam, have been roughly estimated. A modest prestressing force would act favourably for the stresses in the beam, and it would also counteract creep effects for long-term loads. In addition, a natural pre-camber would be achieved, so that this will not have to be dealt with in the manufacturing.

Durability

The FRP and the CRC are very durable materials, as previously mentioned. Hence, the resistance to environmental attack such as chloride penetration, freeze thaw and carbonatisation is very good. However, it has to be validated that the deck surface, especially at the joints between the elements, is absolutely watertight in order to prevent water from penetrating down to the CRC/GFRP interface with possible freeze damage as a result. In addition, the GFRP beams might need to be drained in order to reduce the risk of humidification problems.

Moreover, the reparability of FRP structures is good since they generally ensure localised damage. The repair is simply carried out on site, where the damaged parts are replaced and the structure reinforced by means of new fibres laminated onto it.

Finite Element Analyses

<u>FE Models</u>

The FE analyses conducted are done sequentially on different levels, and with different detailing according to the purpose. First, a two-dimensional analysis of one beam was carried out in order to validate the estimations done so far. Next, the scope was expanded into 3D analyses of one beam, enabling a more realistic FE modelling of the beam. Finally, in order to capture the behaviour of the whole bridge and especially the deck, a global 3D model including all beams and the deck was developed. The 3D models made use of curved shell elements, while all interfaces and joints were modelled with interface elements. The different shells and interfaces of the models were connected with eccentric tyings, i.e. stiff connections. The analyses were carried out using the general finite element program Diana; see [32]. The element mesh of the global 3D model is shown in Figure 6.

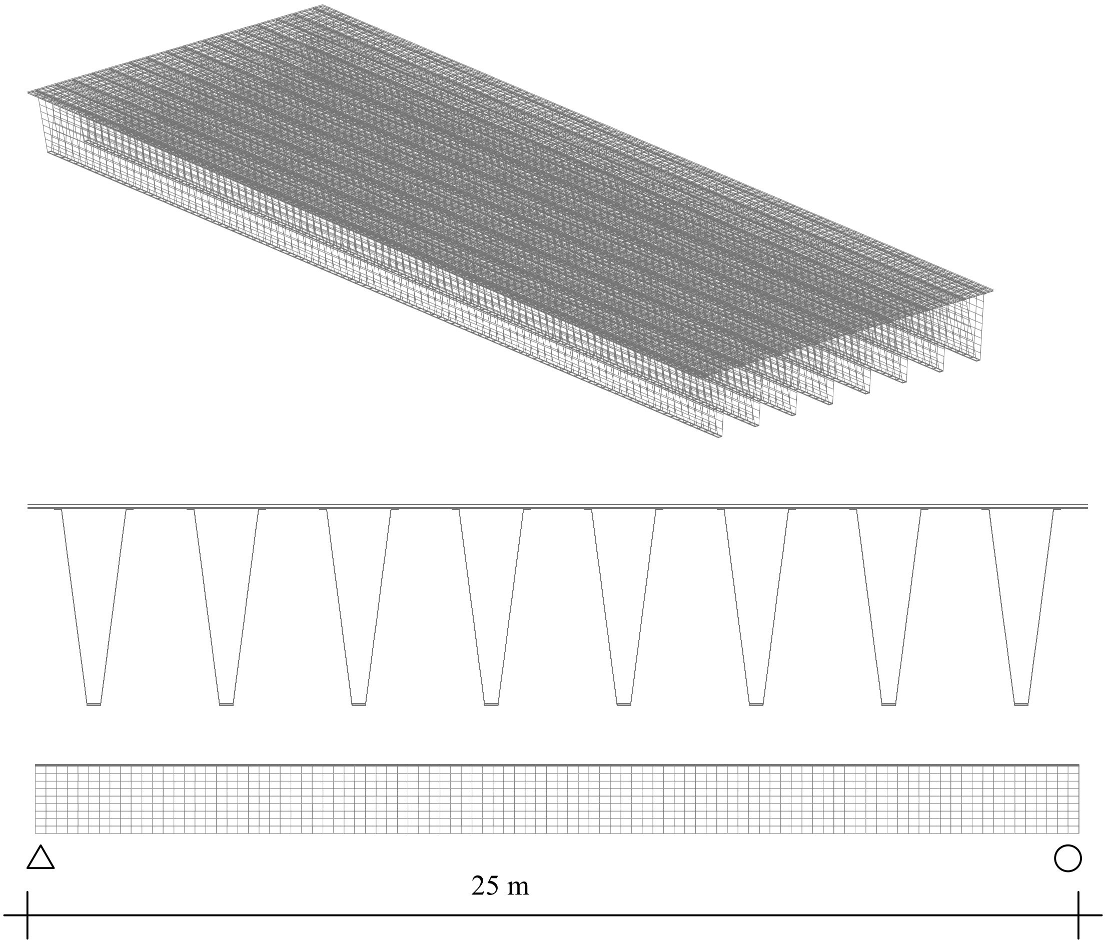

Fig. 6: The element mesh of the global 3D model.

<u>Analyses</u>

Analyses were carried out in both SLS and ULS. In addition, the modes of Eigen-frequency were evaluated and buckling analysis was performed.

The FRP components were modelled with linear elasticity in the analyses, since FRP behave linear elastic until failure.

Results

The results from the FE analyses confirm the conceptual design and demonstrate that the bridge concept corresponds to the code requirements. An example of results is presented in the contour plot in Figure 7.

The design-governing deflection issue were checked. The maximum deflection in SLS received from the analyses was 61 mm, which almost equals the deflection criterion (span-length divided by 400) which gives 63 mm. Hence, the material properties evaluated in the conceptual design seems accurate.

The lowest mode of Eigen-frequency from the analyses was 1.4 Hz.

The issue of optimising the structure needs to be focused on in the continuation of the work. For example, the properties of the deck in the transversal direction must be looked into, since the current analysis indicates that its capacity is on the borderline for the worst load case in ULS.

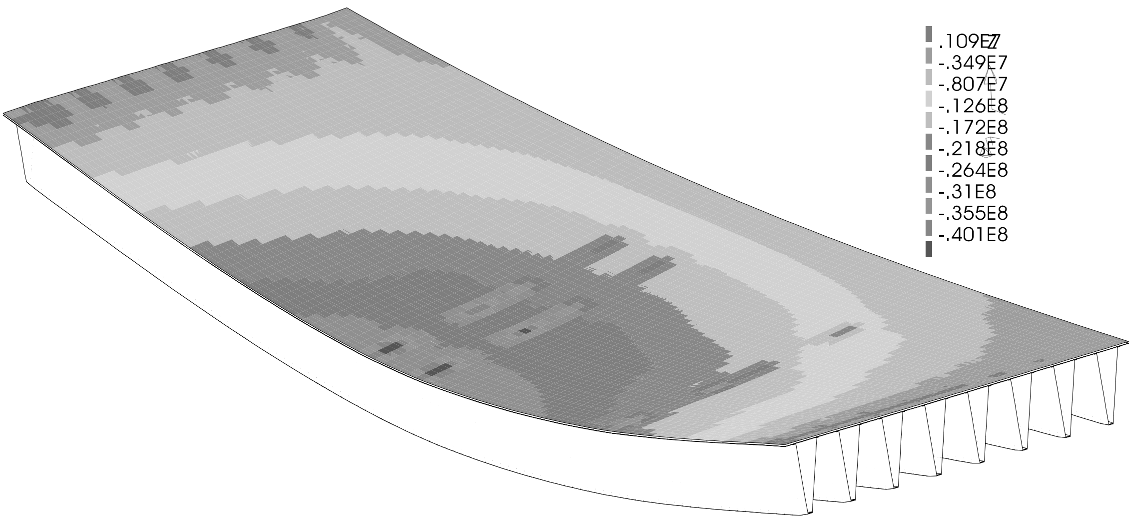

Fig. 7: Contour plot of the CRC concrete in the bridge deck on the deformed shape from the FE analysis. Normal stresses [Pa] in the longitudinal (x) direction at full traffic load (two lanes with three axles each).

Conclusions

The investigations performed, although not covering all details, indicate that the bridge concept can be realised from a technical structural point of view. In addition, the industrial characteristics proposed will ensure efficient production and operation of the bridge. The economic aspects, however, show that there is a need for large production series and decreasing costs for FRP relative to currently conventional construction materials, in order to make the bridge concept competitive in today's market. On the other hand, if life-cycle costs and benefits such as short construction time and improved working environment are taken into account the prospective for the bridge concept looks much better.

In the continuation of the work, there is still much to be done before a prototype bridge can be erected. Hence, detailed design and a set of appropriate tests will be the next steps to take.

Acknowledgement

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Paper VI

Bond between Fibre Reinforced Concrete and Fibre Reinforced Polymers, Experimental Study
Harryson, P.
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Bond between Fibre Reinforced Concrete and Fibre Reinforced Polymers
Experimental Study

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Abstract

An experimental study of the bond between ultra-high-performance steel-fibre reinforced concrete (UHPSFRC) cast upon glass-fibre reinforced polymer (GFRP) sheets, as well as of the bond between epoxy-jointed precast UHPSFRC sheets, has been conducted at Chalmers University of Technology. Both shear and tension tests have been performed. Several surface treatments of the different interfaces have been tested. For the joint interface between UHPSFRC and GFRP the best performance was found for a surface with double-bent steel fibres added to epoxy overspread with sand coating, which were allowed to harden prior to casting the concrete. For the epoxy-jointed concrete sheets the best performance was found for a surface where the smooth "formwork side" was sand-blasted before the epoxy gluing. However, it was concluded that more testing is needed before decisive recommendations can be made. In addition, materials tests of the UHPSFRC have been conducted in both tension and compression.

Key words: Bond, Shear tests, Tension tests, UHPSFRC, GFRP, epoxy.

1. Introduction

The laboratory test and experiments performed at Chalmers University of Technology and presented in this article are a part of the feasibility study of the *i-bridge* concept. In turn, the feasibility study forms a part of a research project concerning industrial bridge engineering. In the first part of the research project (see Harryson (2002)), the focus was on investigating problems in construction causing lack of development and solutions to these. In a wider perspective, the incentive of the research project is to promote development in design and construction of bridges, with an orientation towards industrial construction (i-construction).

The aim of the feasibility study is to illuminate the possibilities offered by new technology, new materials and other advancements in developing concepts of industrial bridge construction. It is an attempt to show how structural engineering can enhance the effectiveness of the overall process and encourage development. The study forms a branch in the *product development* part of the industrial construction process. Hence, the main focus of the feasibility study lies in the technical domain of the process, dealing mainly with technical assessment of structural components.

There are three parts of the study and each part is presented in an article. This article, which is the second part, covers the experimental study and the laboratory tests concerning bond that have been conducted. The test performance and results are presented in more depth in Harryson (2007). The first part of the study (see Harryson (2008:I)) is dedicated to a general description of the bridge concept as well as the initial investigations and numerical analysis that have been carried out. The third and last part (see Harryson (2008:II) deals with the assessment of a test beam, and it covers both the laboratory load testing and finite element analyses.

2. Background

The aim of the experimental tests presented in this article is to evaluate and illuminate certain specially chosen problem areas of the *i-bridge* concept and to assess material parameters for these. The evaluated parameters will be used to validate and calibrate the finite element (FE) analyses. The *i-bridge* concept consists of v-shaped glass-fibre reinforced polymer (GFRP) beams. The beams are reinforced by carbon-fibre reinforced polymer (CFRP) profiles. The deck consists of GFRP plates in composite action with ultra-high-performance steel-fibre reinforced concrete, CRC concrete. CRC stands for compact reinforced composite, which is a commercial available product. More information about the concrete can be found in Nielsen (1995:I) and Nielsen (1995:II).

The essential areas that have been chosen for the tests are the bond between concrete and GFRP in the deck and the bond in the epoxy joints between the prefabricated deck elements. Especially in the first case, the bond in shear is crucial for the performance of the bridge, but the bond in tension is also of interest in both cases. Hence, both shear and tension tests were conducted. The tests were carried out as direct shear and direct tension tests. To be able to simulate the joint behaviour in numerical analyses, not only the failure strength but also the deflections were measured, unlike many other tests of this kind. The experiments were aiming at creating the best bond with the simplest means. Hence, no mechanical devices (e.g. studs etc.) or special geometry (e.g. grooves, indention or ribs) were considered at this stage. A number of different suggested solutions for the two areas of interest were tested in order to be able to make suggestions about jointing technology for best performance. In addition, material parameters for the concrete were needed to verify the capacity and to model the concrete correctly in the FE analysis. Thus, materials tests on the CRC concrete were also carried out.

Rather small dimensions were chosen for the test specimens for the shear and tension tests. This was done mainly in order to get an even stress distribution over the whole test surface and to avoid concentration at edges. It is recognised that it can be argued that this might lead to an overestimation of the bond strength for direct shear tests, since smaller specimens tend to show higher bond strengths due to increased influence from stress concentration at edges; see e.g. Gustavsson (1981) or Carlswärd (2006). The stress concentrations from restrained shrinkage at free edges were studied by Jonasson (1977), indicating the presence of concentrated tension and shear stress distributed over a short length at the edges, for shear about 3 to 5 times the depth of the overlay. Comparing this with the dimensions of the shear specimens, it can be concluded that the shear stress in the test should be reasonably even and embrace a length relatively symmetrical around the peak stress. Moreover, counteracting the size effect is the fact that any malfunction in the bond from some kind of distortion results in more severe effects for smaller specimens.

3. Bonded interfaces

Generally speaking there are three basic mechanisms which account for the bond in these kinds of joints in bimaterial interfaces, namely adhesion, friction and mechanical interlocking; see e.g. Magnusson (2000) with regard to bond of reinforcement in concrete. A lot of research in the different areas of bond has been conducted; although when concrete is included the work mainly focuses on joints between two cementitious layers. A review concerning vital factors in creating good bond between concrete layers can be found in e.g. Carlswärd (2006), where also a summary of different test methods concerning bond is presented. It is stated that there is a large scatter in the results from different test methods. Additionally, the interfacial transition zone (ITZ) plays an important role when cast concrete constitutes one of the adherents, mainly due to the wall effect at the interface; compare e.g. Walter (2005). It has been concluded that the addition of micro-silica has a positive effect regarding the strength of the ITZ and thus improves the bond, see e.g. Löfgren (2005).

Common hypotheses in bond models for joints are the shear friction theory, plasticity-based models, and models based on fracture mechanics. The shear friction theory is the basis for most models found in codes. Among others, CEB-FIP Model Code 90 (see CEB (1993)), Eurocode EC2 (2001), ACI 318 (1999) and the Swedish BBK04 (2004) all have models originating from the friction theory. The code models usually require a compression through the joint or the presence of a clamping effect, while tension is usually not allowed in the absence of reinforcement.

Plasticity models for bond generally assume a cracked sliding interface. Utilising the theory of plasticity for shear strengths in joints gives results similar to those of the less general shear friction theory; see Nielsen (1984).

In models based on fracture mechanics the stress versus crack opening or slip relationship are the essential parameters. For shear there is a difference between elastic brittle behaviour and elastic plastic behaviour, such that in the former case the transferable average shear stress decreases with increasing joint length while in the latter case there is no length influence; see Reinhardt (1989).

In the present study, where the substrate is GFRP in the case of concrete cast joints there are some differences compared with concrete-to-concrete joints. The GFRP substrate has a lower elasticity modulus compared to CRC concrete and CRC has a significantly higher amount of steel fibres providing for a strain hardening response, compared to most steel-fibre reinforced concretes presented in joint studies. Additionally, for example, no problems with laitance or with micro-cracks in substrate have to be accounted for. On the other hand, there are numerous similarities. The same theories and models, possibly with some minor modifications, can be used. Most of the important issues in creating good bond apply, e.g. ensuring a clean surface of the substrate and the essentiality of compacting and curing the concrete. Also applicable is the beneficial roughness of the substrate. Similarly, the same difficulties might be encountered, e.g. restraint shrinkage although the CRC has a somewhat modest shrinkage due to the low water content. The creep that counteracts the shrinkage works in the same way.

In the case of epoxy-jointed concrete surfaces it is the theories of adhesives that are applicable, e.g. from Eurocomp (1996). The design criteria are that the adhesive should be overly strong compared to the adherents and that adhesive failure at interfaces should be avoided. Although not directly applicable in the present tests, there are lots of studies concerning epoxy-bonded interfaces between concrete and FRP in the case of strengthening of existing structures, see e.g. Myers et. al (2007) and Carolin (2003), or Jia et. al (2004) for durability of bonded interfaces. Design guidelines for strengthening concrete structures with FRP can be found in Täljsten (2002).

Given the industrial environment and the proposed production for the concept in the feasibility study, there are vast possibilities to enhance the properties of the joints, i.e. creating an engineered surface.

4. Test interfaces

There are several possible solutions and suggestions to create bond in shear and tension, as been mentioned. The choices of different test surfaces for the material interfaces between CRC and GFRP and for the epoxy-glued interfaces between two CRC concrete surfaces are presented in Table 1. Some of the means evaluated to create good bond is common practice or taken from the literature, while others seems novel. For example sand coating in hardened epoxy is a common feature to enhance the bond and e.g. Schaumann et. al (2006) tested the idea of applying fresh epoxy just before casting the concrete, while the variant with additional specially double-bent steel fibres added to the epoxy before spreading the sand is believed to be novel. Some of the surfaces of the tested interfaces are shown in Figure 1.

A test sheet was manufactured for each of the variants of surfaces mentioned in Table 1. The dimensions of the sheets were 400x400 mm^2 with a thickness of approximately 60 mm (approximately 30 mm CRC concrete or GFRP on each side of the interface). For the shear tests, almost cubic test specimens with the dimensions approximately 50 x 50 x 60 mm^3 were sawn from the sheets with a diamond cutting edge. For the tension tests, cylindrical specimens were core-drilled from the sheets, with an inner core diameter of 50 mm down to the interface to be tested and an outer core diameter of 100 mm through the whole sheet.

Table 1. Summary of the test interfaces showing surface treatments of GFRP prior to casting and of CRC prior to epoxy-gluing.

Interfaces between CRC and GFRP	**Epoxy-glued interfaces**
Surface B: - no treatment but cleaning of the GFRP surface **Surface C:** - applying epoxy on the surface just before casting **Surface D:** - as above, but with an additional quartz sand coating **Surface E:** - sand coating in epoxy which was allowed to harden before casting **Surface F:** - like the preceding, but with additional specially double-bent steel fibres added to the epoxy before spreading the sand **Surface G:** - special GFRP surface with non-bonded glass fibres **Surface H:** - special GFRP surface with a “peel-ply” on top; the ply was removed prior to casting, resulting in a rough sandpaper-like surface	**Surface A:** - as a reference test two GFRP surfaces joined by epoxy adhesive were tested **Surface I:** - the smooth “formwork side” was sand-blasted before the epoxy gluing **Surface J:** - retardant was applied in the formwork and aggregates and steel fibre were subsequently revealed by rinsing with high water pressure prior to the epoxy gluing **Surface K:** - special bent steel fibres, with one part cast into the concrete, reinforced the epoxy joint

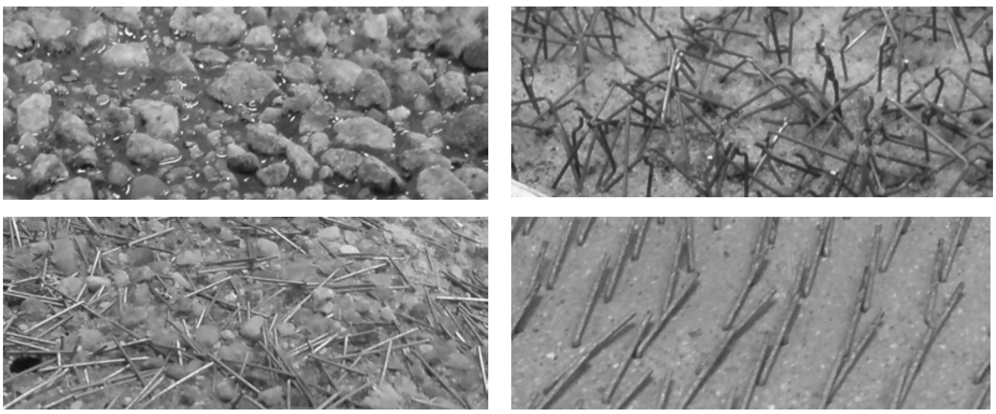


Figure 1. Details of some of the surfaces of the tested interfaces before casting of concrete or epoxy-gluing. Above left is surface D, above right is surface F (before removal of loose sand), below left is surface J and below right is surface K.

It can be noticed that it was not possible to cut out any test specimens for surfaces B and G, since the interfaces separated due to lack of adhesion. Consequently, there are no results to present for these surfaces.

5. Shear tests

5.1 Performance

A series of at least three test specimens was sawn from the test sheet for each of the interfaces to be tested. Each specimen was mounted with epoxy glue to special steel angles in order to produce a monolithic specimen for testing. The steel angles were firmly tied to a plane, thick and stiff steel sheet, ensuring that the corresponding sides of the monolithic specimen became parallel and at the same time allowing for the adjustment of the cut specimen into the right position. The principles for the test and the set-up are shown in Figure 2. The load and displacement were measured with a load cell and through two LVDT gauges mounted on the specimen.

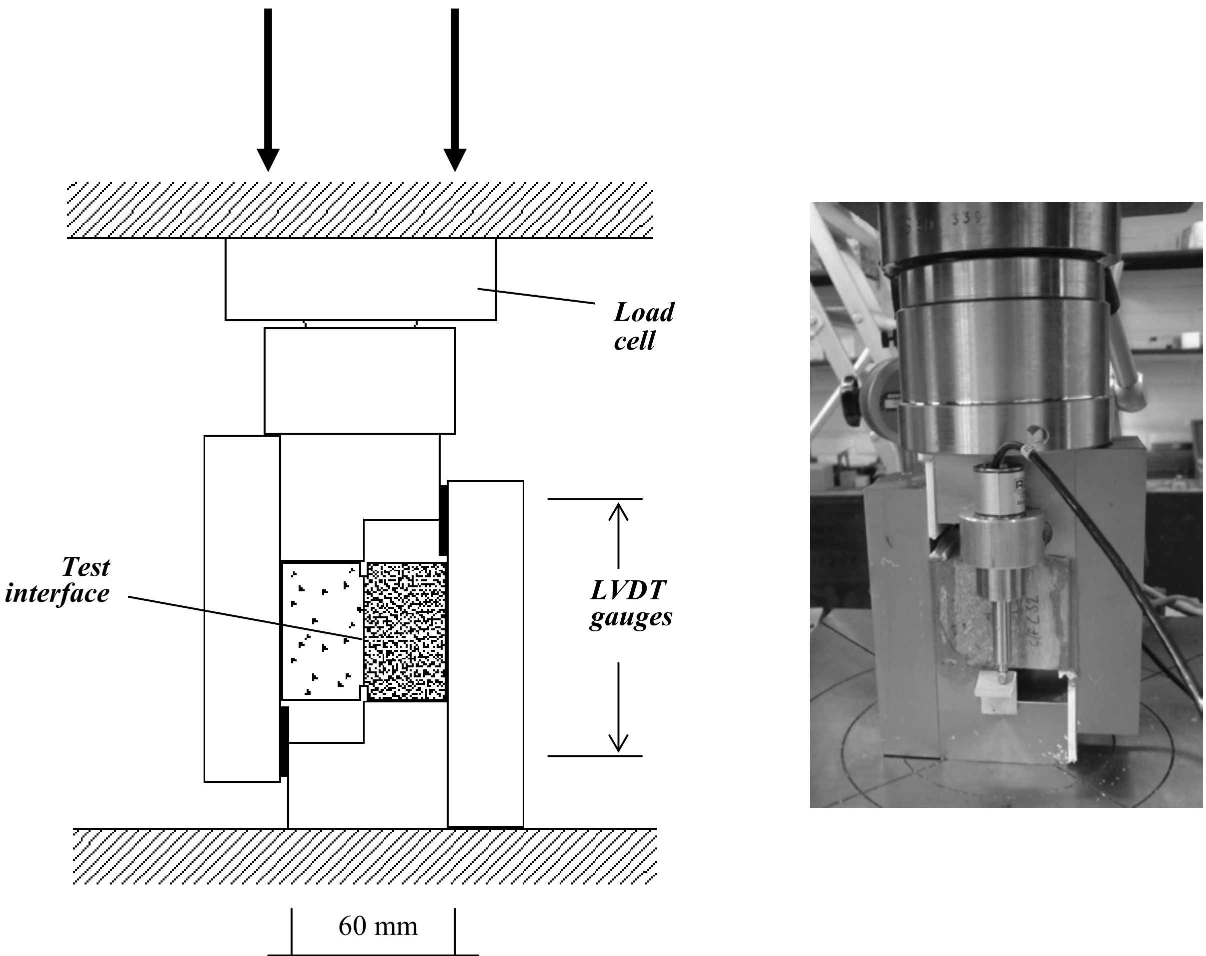


Figure 2. Test principles and set-up for the shear test.

5.2 Results of shear tests

A summary of failure loads and corresponding calculated average shear stress are presented in Table 2. Diagrams showing a summary of the test results regarding calculated average shear stress vs. displacement are shown in Figure 3 for the interface between GFRP and concrete along with Figure 4 for the epoxy-jointed concrete surfaces.

Table 2. Summary of results from the shear tests. Denotation c stands for concrete and f stands for fibre-reinforced polymer. The first capital letter defines the test series and the second stands for shear test.

Specimen	Failure load (kN)	Average load at failure (kN)	Calculated Shear stress (MPa)	Average calc. Shear stress (MPa)
f/f AS4	22,80	21,21	12,74	12,07
f/f AS5	22,96		13,21	
f/f AS6	17,87		10,28	
c/f CS1	7,10	7,46	3,57	3,66
c/f CS2	8,11		3,90	
c/f CS3	7,16		3,52	
c/f DS1	5,41	7,07	2,95	3,37
c/f DS2	8,12		3,99	
c/f DS3	6,45		3,17	
c/f ES1	5,41	6,11	2,70	3,08
c/f ES2	4,73		2,41	
c/f ES3	8,18		4,12	
c/f FS1	13,79	10,75	6,81	5,29
c/f FS2	10,73		5,26	
c/f FS3	7,72		3,79	
c/f HS1	11,68	9,74	5,78	4,79
c/f HS2	8,89		4,29	
c/f HS3	8,64		4,29	
c/c IS0	21,84	20,76	10,93	10,31
c/c IS1	17,76		8,82	
c/c IS2	21,62		10,67	
c/c IS3	21,62		10,81	
c/c JS1	13,72	15,27	7,07	7,75
c/c JS2	12,86		6,63	
c/c JS3	19,23		9,54	
c/c KS1	20,20	18,37	9,78	8,81
c/c KS2	17,43		8,55	
c/c KS3	20,20		8,09	

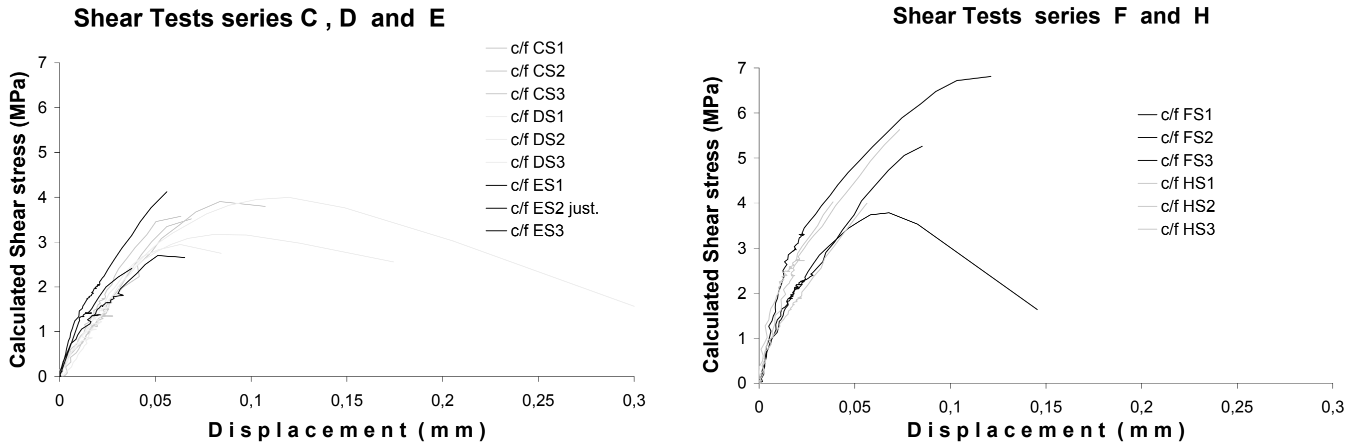


Figure 3. Test results for all interfaces between concrete and GFRP, calculated shear stress vs. displacement. To the left series C, D and E, to the right series F and H.

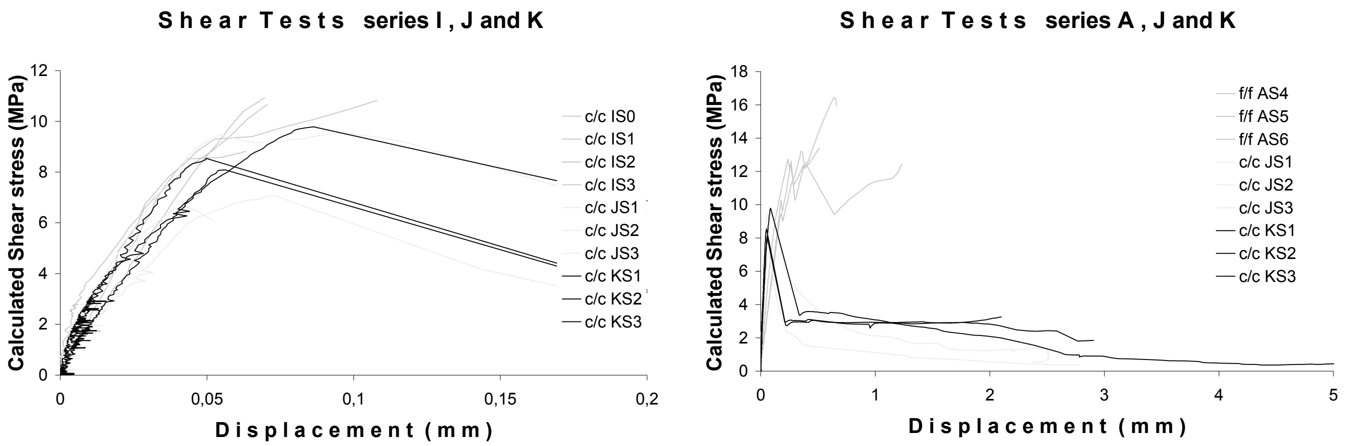
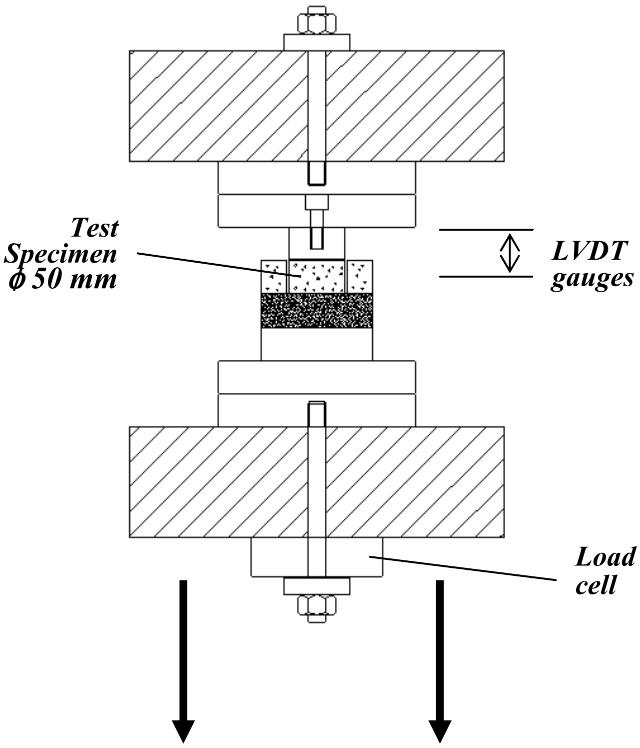


Figure 4. Test results for all epoxy-jointed surfaces, calculated shear stress vs. displacement. To the left series I, J and K, to the right series A, J and K. Notice the different scales.

6. Tension Tests

6.1 Performance

A series of at least three specimens were core-drilled from the test sheets. Each of the specimens was glued with epoxy to circular steel plates. The mounting was done in the test machine in order to ensure parallelism of the steel plates and that mounting would be done without eccentricities. The principles and the set-up for the test are presented in Figure 5. The load and displacements were measured with a load cell and through three LVDT gauges mounted on the specimen.



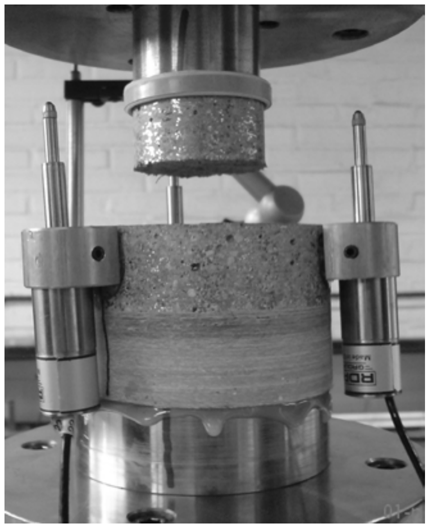


Figure 5. Test principles and the set-up for the tension tests (after loading).

6.2 Results of the tension tests

A summary of the failure loads and their corresponding calculated average tension stress is presented in Table 3.

Diagrams showing a summary of the test results regarding calculated average tension stress vs. calculated average displacement in the interface are shown in Figure 6 for the interface between GFRP and concrete, along with Figure 7 for the epoxy-jointed surfaces.

Table 3. Summary of results from the tension tests. Same denotation as in Table 2.

Specimen	Failure load (kN)	Average load at failure (kN)	Calculated Tension stress (MPa)	Average calc. Tension stress (MPa)
f/f AT0	9,05	13,16	4,38	6,37
f/f AT1	12,07		5,82	
f/f AT2	16,70		8,07	
f/f AT3	14,80		7,20	
c/f CT1	2,01	2,67	0,97	1,29
c/f CT2	2,69		1,30	
c/f CT3	3,32		1,61	
c/f DT1	2,47	2,79	1,20	1,35
c/f DT3	3,15		1,53	
c/f DT4	2,76		1,33	
c/f ET0	4,96	3,59	2,40	1,74
c/f ET1	4,31		2,08	
c/f ET2	1,76		0,83	
c/f ET3	4,55		2,21	
c/f ET4	2,43		1,17	
c/f FT2	7,13	8,30	3,45	4,02
c/f FT3	9,17		4,45	
c/f FT4	8,59		4,15	
c/f HT1	4,77	4,36	2,31	2,11
c/f HT2	4,77		2,31	
c/f HT3	4,16		2,01	
c/f HT4	3,73		1,80	
c/c IT0	12,21	11,33	5,91	5,48
c/c IT1	12,68		6,13	
c/c IT2	9,42		4,56	
c/c IT3	11,01		5,33	
c/c JT2	13,78	12,95	6,67	6,27
c/c JT3	13,07		6,34	
c/c JT4	12,00		5,80	
c/c KT1	6,43	10,07	3,12	4,88
c/c KT2	12,34		5,99	
c/c KT3	11,45		5,54	

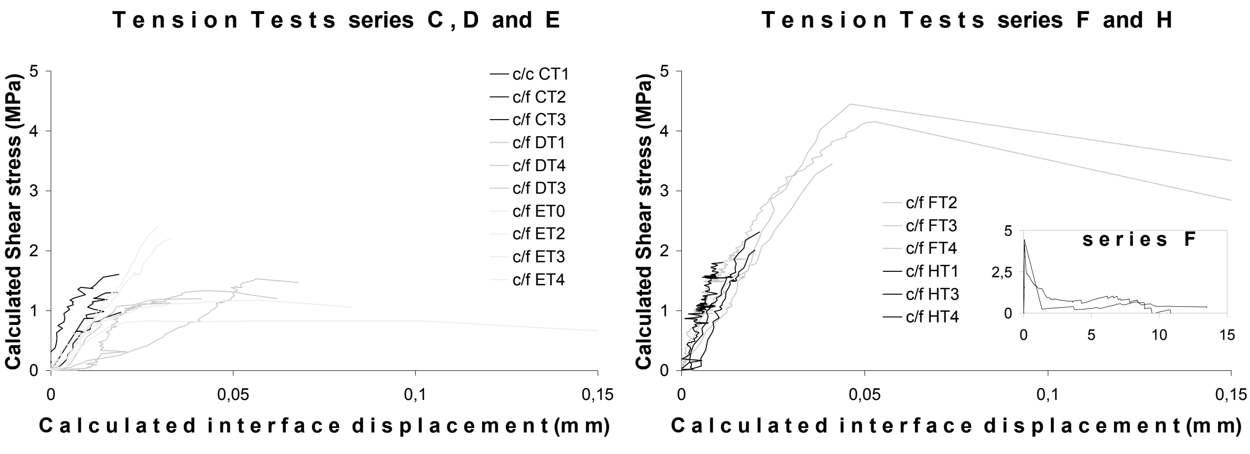


Figure 6. Test results for the interfaces between concrete and GFRP, calculated average tension stress vs. calculated average displacement in the interface. To the left series C, D and E, to the right series F and H (with cut-in descending branch for F).

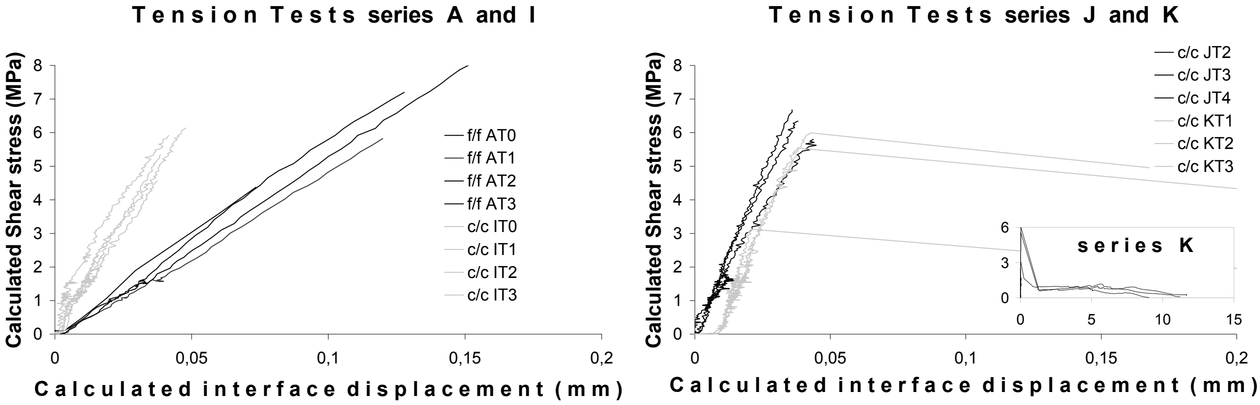


Figure 7. Test results for the epoxy-jointed surfaces, calculated average tension stress vs. calculated average displacement in the interface. To the left series A and I, to the right series J and K (with cut-in descending branch for K).

7. Tension testing of CRC

7.1 Performance

A series of six dog-bone-shaped specimens was cast for tension testing of the CRC concrete. The dimensions of the central measurement field of the dog bones were 100x50x30 (length x width x thickness). Each specimen was mounted with epoxy glue to special steel parts which were fixed in the test machine. Two strain gauges were mounted on the short side of the specimens. The principles and the set-up for the test are presented in Figure 8. The load, strain and displacement were measured with a load cell and through the two strain gauges and two LVDT gauges mounted on the specimen.

7.2 Results

A summary of crack load and the corresponding calculated average tension stress along with the failure load and the corresponding average calculated tension stress are shown in Table 4. The crack load has been estimated from the test diagrams.

Diagrams which present summaries of the test results for test specimens DB1, DB2 and DB4 are shown in Figure 9. The rest of the specimens are not presented because the cracking of the specimens made the graphs difficult to interpret, since the failure was localised in cracks outside the measurement field. The failure loads were lower than expected, which might be due to uneven distribution of steel fibres in the concrete; this is further discussed in section 9.

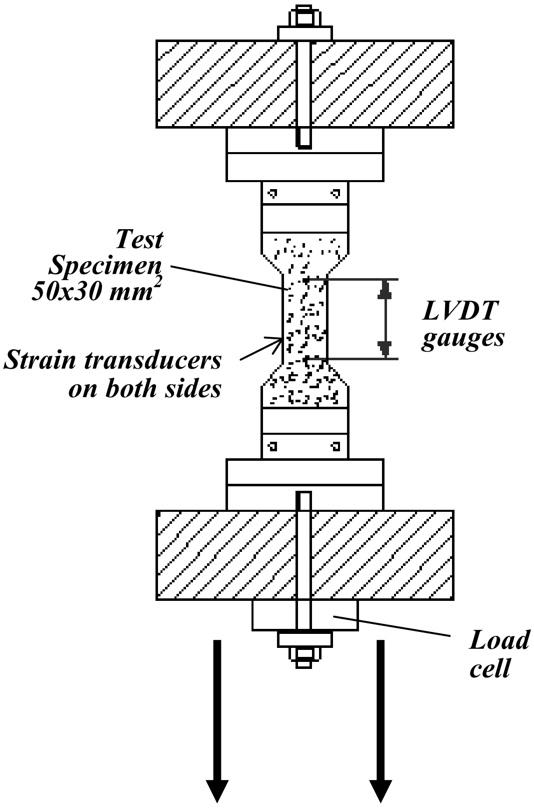


Figure 8. The principles and the set-up for the tension tests of CRC.

Table 3. Summary of results from tension test of CRC concrete.

Specimen	Crack load (kN)	Average Crack load (kN)	Calc. Tension stress (MPa)	Average calc. Tension stress (MPa)	Failure load (kN)	Average load at failure (kN)	Calc. Tension stress (MPa)	Average calc. Tension stress (MPa)
DB1	11,8		8,2		11,88		8,24	
DB2	10,9		7,8		11,05		7,90	
DB3	11,7	11,4	8,0	7,9	13,11	12,43	8,96	8,55
DB4	12,7		8,7		14,42		9,85	
DB5	11,3		7,7		13,56		9,25	
DB6	10,3		7,0		10,51		7,14	

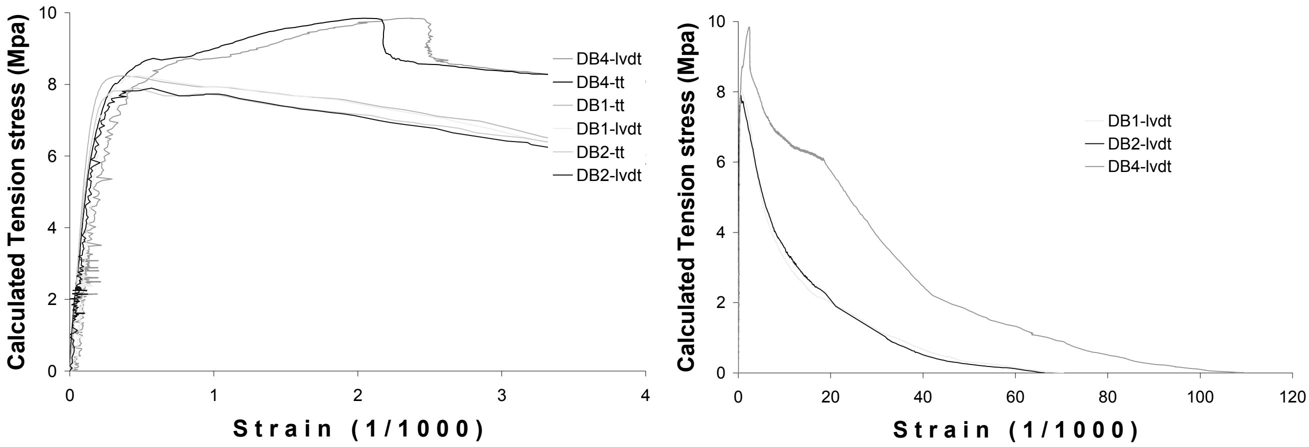


Figure 9. Test results for specimens DB1, DB2 and DB4, calculated tension stress vs. strain. To the left average measurements for strain gauges and LVDT gauges (calculated strain), to the right the descending branch for the LVDT gauges. Notice the different scales.

8. Compression testing of CRC

Three series of three cylinders each were also cast for compression tests and evaluation of elasticity modulus (on one series). The dimensions of the cylinders were ϕ100x200 mm^2. A summary of the results from the compression testing of CRC concrete and the evaluation of the modulus of elasticity are presented in Table 5.

Table 5. Summary of results from the compression tests of CRC concrete.

Cylinder	Stress at failure f_{cc} (MPa)	Average f_{cc} (MPa)	Modulus of Elasticity E_c (GPa)	Average E_c (GPa)	Age (days)
1:1	145,0				
1:2	147,2	147,9			29
1:3	151,6				
2:1	157,7		52,16		
2:2	153,5	157,4	55,50	54,8	56
2:3	161,0		56,80		
3:1	162,2				
3:2	165,1	153,7			63
3:3	133,9				

9. Discussion

The difficulties encountered with regard to keeping an even stress over the whole test surface is primarily due to the composition of the test surfaces which by their nature encompass parts that cause stress concentrations. But also, due to the small dimensions of the test specimens stress concentrations will have a significant effect. However, this effect can be even worse for large specimens. Thirdly, although designing the tests so as to keep an even distribution of loads throughout the tests, there is always a possibility that the tests will be influenced by small moment effects, especially during the course of failure. Evaluating the difference in the measured displacement between the different LVDT gauges in each test gives an estimation of the problem. There are large differences in the displacements for most of the tension tests and for about 50% of the shear tests. Not surprisingly, there has been an obvious trend that tests with uneven displacements show a lower failure load than those with even displacements. Consequently, this affects the scatter in the test, which is rather large in some series.

Thus, the average stress presented cannot be taken for the real failure stress, unless in the case of even displacements where it can be seen as a limit for the local failure stress. For the same reason, care must be taken when ranking the different series.

Another important factor to comment upon is the distribution of steel fibres in the CRC concrete. From the results of the tension tests of the concrete, it was inferred that the distribution of steel fibres might not be adequate. All dog bone specimens went to failure at a significantly lower load than expected, while all but one did not show the strain hardening response that was supposed. The explanation for this is an uneven distribution of fibres due to mistakes when mixing and casting the concrete. Hence, a too large mixing batch and excessive vibration of the dog-bone specimen seems likely to have led to this problem. This can be concluded when calculating the fibre efficiency factor for the concrete $\boldsymbol{n_b}$; see e.g. Löfgren (2005). This factor is 1.0 for one-dimensional distribution, and 0.64 and 0.5 for two- and three-dimensional distribution respectively. The geometry of the dog bone specimens should account for a distribution somewhere in between two- and three-dimensional. Calculating the amount of fibres in the failure surface for specimen DB1 gave $n_b = 0{,}32$. Although the scatter can be quite significant, as reported e.g. in Carlswärd (2006), this clearly indicates that there is not a sufficient amount of fibres present in the failure surface.

Still, this is not believed to affect the results of the shear and tension tests significantly, since the crack stress of the CRC is higher than the presented calculated average stresses, except for the shear stresses for the epoxy-glued CRC sheets which are in the same range. In addition, when examining the failure surfaces and the cut surfaces of the specimens they do not draw attention to any effects from the uneven distribution of fibres in the CRC. The effects in the materials tests for the CRC are more significant, however, especially in the tension tests.

Effects of shrinkage and creep have not been studied in the tests, although there is, of course, a certain amount of shrinkage that has taken place in the test sheets from the time of casting until the specimens were cut out.

The ratio of shear to tension stress can be found to be about 2 from the literature on concrete-to-concrete joints: see e.g. Carlswärd (2006). In the present study this ratio varies from 1.2 to 2.8, while it must be kept in mind that it is engineered surfaces that are tested. Hence, it can be stated that the ratios are of the same magnitude.

Summing up the results from the shear and tension tests, the following brief comments can be made while a more elaborative discussion can found in Harryson (2007).

For test sheet c/c A (two GFRP sheets jointed by epoxi), which were reference tests, it can only be concluded that the epoxy joint is stronger than the GFRP-sheets.

For test sheets c/f B – H (interface between CRC cast upon GFRP) the results differed a great deal. It was expected that the test sheet c/f B (no treatment but cleaning of the GFRP surface) would give low bond values and the delamination of the joint prior to testing revealed that the bond strength actually was close to zero. The same thing was expected for test sheet c/f G (special GFRP surface with non-bonded glass fibres) since the manufacturing was abortive with few fibres actually protruding from the surface. When comparing the test sheets c/f C (applying fresh epoxy on the surface just before casting) and c/f D (fresh epoxy with an additional quartz sand coating) one can conclude that their performances were about equal. It was noticed that the failure surfaces of both shear and tension test specimens from both sheets showed traces of what is believed to be laitance, but still the results were a bit surprising since the epoxy employed in the joints, among other things, was supposed to bond to fresh concrete. Observed for the shear test series from sheet D is a more ductile response compared to the other test series.

If test sheet c/f E (sand coating in epoxy which was allowed to harden before casting) is compared to the two sheets above, it can be concluded that sheet E performs somewhat better in shear and bit poorer in tension. There is also a large scatter in the results probably due to the amount of added sand not being optimised with regard to the different fractions. An optimisation of the sand additive would most likely result in a better bonding performance and, moreover, eliminate the durability doubts that arise for the current composition.

The best performance for the concrete over-layered surfaces was achieved for test sheet c/f F (like the preceding, but with additional specially double-bent steel fibres added to the epoxy before spreading the sand), as expected. Compared to test sheet E, the failure surface is transferred down deeper into the epoxy layer. The residual capacity seen in the tension test is believed to be fictitious and probably due to part of the steel fibres protruding outside the test surface. The same situation as for sheet E applies with regard to the optimisation of the sand content. In addition, it could be interesting to study how much the amount of the double-bent fibres can be decreased while still arresting a failure surface in the epoxy. In the current tests, the addition of bent fibres was 12500 per m^2, which theoretically corresponds to an upper limit for the tension failure stress of 7.8 MPa (calculated as the failure load for the fibres per area unit).

The test series from sheet c/f H (special GFRP surface with a "peel-ply" on top; the ply was removed prior to casting, resulting in a rough sandpaper-like surface) showed better performance than all other series apart from test sheet F, which is especially interesting in view of the simple arrangements. An optimisation of the "peel-ply" type could perhaps enhance the performance even more. The test series also showed a reasonable low scatter.

The test results were relatively equal for the three test sheets with epoxy-jointed surfaces. Sheet I (where the smooth "formwork side" was sand-blasted before the epoxy gluing) demonstrates even results with a low scatter for both shear and tension tests.

For test sheet J (where retardant was applied in the formwork and aggregates and steel fibres were subsequently revealed by rinsing with high water pressure prior to the epoxy gluing), somewhat lower test results were achieved in the tension test compared to sheet I, possibly due to a lowered concrete quality near the surface due to the water rinsing.

The amount of added steel fibres for sheet K (special bent steel fibres, with one part cast into the concrete, reinforced the epoxy joint) was, as for sheet F, 12500 fibres per m^2, similarly constituting a theoretical upper limit for the tension failure stress of 7.8 MPa. It can be said that the method performed according to expectations. The residual strength demonstrated in both the shear and the tension tests is probably due to a certain toughening effect from the fibres being pulled out or sheared off respectively. In tension, however, a part of the residual strength is probably a fictitious effect similar to that of sheet F. The method would perhaps better suit the situation with concrete-to-concrete joints, the fibres being straight in that case and the protruding part being cast in.

10. Conclusions

Some of the test results show a somewhat high scatter, primarily due to uneven stress distribution over the test surfaces caused by stress concentrations. Hence, the presented average shear and tension stresses cannot be taken for the real failure stresses, unless in the case of even displacements where they can be seen as a limit for the local failure stresses. For the same reason, care must be taken when ranking the different series, especially when their average results are close.

There was an uneven distribution of steel fibres in the cast dog-bone specimen for tension tests of the CRC, caused by insufficient mixing and excessive vibration. This should not have affected the shear and tension tests, however, since the mobilised stresses in most cases are below the crack stress of the CRC concrete. But the effects in the materials tests for the CRC are more significant, especially in the tension tests.

Since no optimisation has been done until this stage, there is still some work to be done in order to be able to make a choice of surface treatment for the CRC concrete cast upon GFRP sheets. It is possible, however, to exclude some of the test surfaces already at this point. These are surface B (no treatment but cleaning), surface C (epoxy applied on the surface just before casting) and surface G (special GFRP surface with non-bonded glass fibres). The most suitable surface treatment for the test beam seems to be surface H (special GFRP surface with a “peel-ply” to be removed prior to casting), to judge from the tests. It possesses a sufficient capacity and has a low distortion among its test results.

For the CRC sheets joined by epoxy, surface I (the smooth formwork surfaces being sand-blasted before jointing) appears to be the simplest and probably the best alternative.

Acknowledgement

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Paper VII

Laboratory test and finite element analyses of a prototype bridge beam
Harryson, P.
Submitted to *Nordic Concrete Research*

Laboratory test and finite element analyses of a prototype bridge beam

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Abstract

A laboratory load test and finite element analyses of a prototype bridge beam have been conducted at Chalmers University of Technology as a part of the feasibility study of a novel bridge concept, the *i-bridge*. The beam consisted of v-shaped glass-fibre reinforced polymer (GFRP) webs reinforced by a carbon-fibre reinforced polymer (CFRP) profile, with a deck plate made of GFRP in composite action with ultra-high-performance steel-fibre reinforced concrete (UHPSFRC). The FRP parts are jointed by means of epoxy adhesive, while the concrete is simply cast upon the specially roughened GFRP surface. The load test was done in four point bending and it confirmed the predicted structural behaviour from the finite element (FE) analyses. However, it was concluded that more testing will be needed in the forthcoming work before decisive recommendations about the performance can be made.

In addition, materials tests of the UHPSFRC have been conducted, both in tension and compression.

Key words: Prototype beam, Four point bending, FE analysis, UHPSFRC, GFRP, CFRP, epoxy.

1. Introduction

The laboratory test and finite element (FE) analysis of a prototype bridge beam conducted at Chalmers University of Technology that are presented in this article form a part of the feasibility study of the *i-bridge* concept. The study in turn is part of a research project concerning industrial bridge engineering. The aim of the study is to illuminate the possibilities given by new technology, new materials and other advancements when developing concepts of industrial construction (i-construction), and particularly industrial construction of bridges. In an overall perspective, the incentive of the research project is to encourage development in design and construction of bridges, with a more direct view towards i-construction.

The feasibility study is divided into three parts and each part is presented in an article. This article is the third and last part. The first part, Harryson (2008:I), is dedicated to a general description of the concept as well as the initial investigations and numerical analyses that have been undertaken. The second part, Harryson (2008:II), concerns an experimental study of bond in the bimaterial interfaces in the bridge deck, including laboratory tests.

2. Background

The feasibility study, of which the third and last part is presented in this paper, is an effort to show how structural engineering can promote enhancement of efficiency in the overall process and support development. The study outlines a branch in the *product development* part of the industrial construction process; compare Figure 1. Still, the input to the overall process can be considerable even though this is not the focus of the study. Hence, the main focus of the present study lies in the technical domain of the process, concerning mainly technical assessment of structural elements.

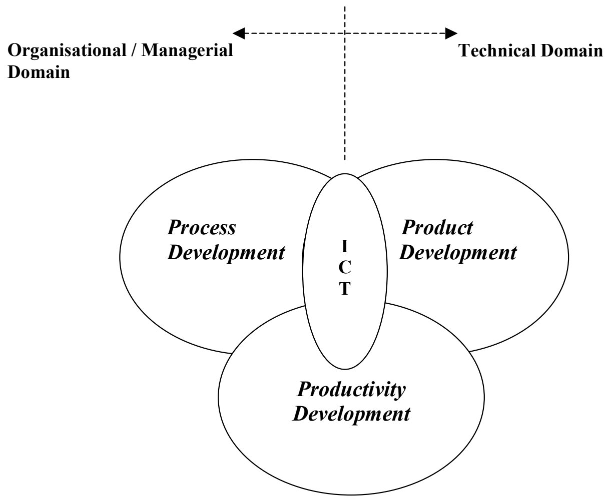


Figure 1. The cornerstones of Industrial Bridge Construction – the three P's. (From Harryson (2002))

The aim of the laboratory load test and FE analyses presented in this article is to assess a prototype bridge beam of the *i-bridge* concept. The *i-bridge* concept consists of v-shaped glass-fibre reinforced polymer (GFRP) beams. The beams are reinforced by carbon-fibre reinforced polymer (CFRP) profiles. The deck consists of GFRP plates in composite action with ultra-high-performance steel-fibre reinforced concrete (UHPSFRC), CRC concrete. CRC stands for compact reinforced composite, which is a commercially available product. More information about the concrete can be found in Nielsen (1995:I) and Nielsen (1995:II). The results from the load test are presented more in depth in Harryson (2008:III).

3. Test objectives and limitations

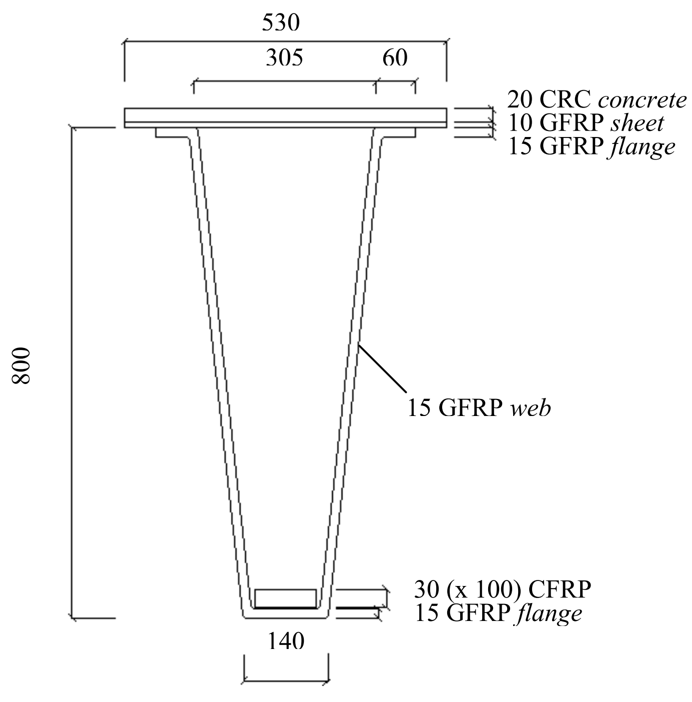
In developing a novel bridge concept, there is a range of tests that are essential and valuable to validate the performance. Thus, both materials tests and tests of details are needed, but also tests of the whole structure or large parts thereof. There is, however, a need to limit the areas of interest to the most essential issues in feasibility studies. In addition, it is important not to forget that the tests performed in feasibility studies are initial, and that improvements may be needed at a later stage.

The main focus of the load test is to demonstrate the structural performance with special attention to the composite action in the deck, to follow and infer the failure mode, and to validate the correctness of the numerical tools used. Some similarities in the approach to testing and analysis can be found in e.g. Fam (2006) and Moussa & Zhao (2004).

Of course, since this is a unique test the results cannot be statistically verified. Additionally, the test does not tell us anything about the behaviour in the transverse direction, i.e. how the deck performs in that direction. Moreover, since the test beam is scaled down relative to the real structure, there might be a discrepancy in behaviour between the two. However, this is checked to some extent by comparing the numerical analyses of the two structures.

4. The prototype test beam

The beam prototype was composed similarly to the beams in the *i-bridge* concept (compare Harryson (2008:I); only its dimensions had been scaled down in some aspects. The span of the test beam was 5.0 m, whereas the span in the concept is 25 m. The total height was lowered from 1580 mm to 820 mm and the thickness of the CRC concrete was decreased from 70 mm to 20 mm. The width of the deck and the distance between the webs were decreased correspondingly, while it was decided to keep the thicknesses of the GFRP and the capacity of the CFRP intact as in the concept. This was mainly because an aim of the test was to validate the composite action between concrete and GFRP. Hence the FRP parts would not be critical, and testing the phenomenon of composite action could more easily be achieved in the test. In addition, the tolerances of the FRP components justified a certain margin. But a further practical reason was to be able to handle the structure and to perform the test with a reasonable magnitude of load. A cross-section and a picture of the prototype beam are shown in Figure 2.



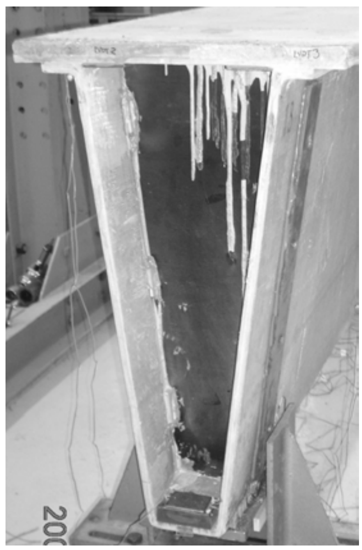


Figure 2. The cross-section of the prototype beam (to the left, dimensions in mm) and a picture showing the beam after casting of the concrete in the deck (to the right).

The GFRP consisted of E-glass in polyester matrix. Material properties for the GFRP were evaluated by assuming a fibre volume fraction of 50%. All laminates used were essentially bi-directional with only a small portion of the fibres (about 20%) in the diagonal directions. In addition, the volume fraction of fibres was differentiated across the thickness of the laminate, so that the amount of longitudinal fibres was increased in the middle half while the amount of transverse fibres was increased correspondingly in the two outer fourths close to the skin. The CFRP was made from prepreg of unidirectional high strength PAN carbon fibres embedded in epoxy resin. The volume fraction of carbon fibres was 60%. The FRP parts were joined by means of epoxy adhesive. The amount of steel fibres used in the CRC concrete was 6% by volume.

The surface of the GFRP deck sheet consisted of a special surface with a ”peel-ply” on top which was removed prior to casting, resulting in a rough sandpaper-like surface. This was the surface with one of the best performances in the shear and tension bond tests; see Harryson (2008:II). The FRP beam was manufactured in a shop while the CRC concrete was cast in the laboratory. The GFRP components were made by hand lay-up and vacuum bagging using polyester resin, and the CFRP were made from epoxy prepreg as mentioned.

Crossbeams were cast at beam-ends above supports when casting the deck. For the reason of load transfer between the crossbeam and the GFRP web, double steel plates with welded steel studs on the inner plate – to be cast into the crossbeams – were bolted and glued by means of epoxy adhesive to the webs. In addition, bearing plates of steel were epoxy-glued to the bottom flange at supports.

A few small shrinkage cracks in the transverse direction of the CRC surface were observed some weeks after casting. These were probably induced already in the early stage after casting (autogenous shrinkage), since it had been concluded that stresses from shrinkage would be much lower than the cracking stress of the hardened CRC. The beam was load-tested 49 days after casting.

5. Finite element analysis

5.1 Modelling

Numerical analysis has been carried out to simulate both the test beam and a full-size beam spanning 25 m. This has been done in order to ensure that the test beam behaves correspondingly when loaded to failure, and that the phenomena observed in the test also are those that can be expected in the full-scale structure.

The analysis was carried out with the general finite element program Diana; see TNO (2005). Both FE models were done in 3D using curved shell elements, while all material interfaces and epoxy joints were modelled with interface elements. The different shells and interfaces of the models were connected with eccentric tyings, i.e. stiff connections. The materials models used allowed for nonlinear behaviour, although the FRP acts linear elastic until failure. The FRP were modelled with the Hoffman failure criteria, while the concrete was modelled based on smeared cracking and total strain, and the plasticity model of Thorenfeldt accounted for the non-linearity of concrete in compression. In some analyses the concrete were modelled with isotropic plasticity and von Mises failure criteria. Additionally, the interface elements were modelled with multi-linear relationships; compare TNO (2005). The modelling of the FRP could have been enhanced for example by the use of fracture mechanics, compare e.g. Moore (2004), but this did not seem neccesary since the strains and stresses in the FRP were expected to be fairly modest due to the down-scaling of the beam. For the same reason the chosen failure criteria seems appropriate for the present study, while there are many failure criteria that can be adopted for FRP, compare e.g. Hinton et. al (2004).

Material properties for FRP were calculated on the basis of the properties of the fibres and the matrix from the manufacturers. Materials data for the joint epoxy were received from Ingles & Mendoza (2004) and from the manufacturer. In addition, evaluation of the bond tests, see Harryson (2008:II), gave supplementary information. Input data for the CRC were fetched from previous material tests as well as from Nielsen (1995:I). For the interfaces between CRC and GFRP, parameters were evaluated from information about the response in shear and tension from the bond tests.

Since no materials testing could be conducted for the FRP parts, there was a certain level of uncertainty for the calculated material properties. However, due to the chosen scaling of the beam, this was not believed to be critical except for the shear resistance in the GFRP webs. In addition, it was not possible to assess information about the shear capacity of the matrix in the GFRP, so estimation was necessary.

The assumed material parameters used in the FE simulations are presented in Table 1. The element mesh of the test beam model is shown in Figure 3.

Table 1. Material properties used in the FE analyses.

CFRP	**GFRP**	**CRC**	**Epoxy (interface)**	**CRC - GFRP interface**
E_x = 135 GPa	E_x = 21,1 GPa	E_c = 54,8 GPa		
E_y = 10 GPa	E_y = 21,1 GPa	ν = 0,24	<u>*normal direction*</u>	<u>*normal direction*</u>
E_z = 10 GPa	E_z = 10,7 GPa	f_{cc} = 150MPa	D_{11}= 53 GPa/m	D_{11}= 100 GPa/m
G_{xy} = 5,0 GPa	G_{xy} = 5,1 GPa	f_{ct} = 14 MPa	σ_{nt} = 8 MPa	σ_{nt} = 2 MPa
G_{yz} = 3,0 GPa	G_{yz} = 5,1 GPa	$f_{ct,cr}$ = 7 MPa	δ_{nt}= 0,15 mm	δ_{nt}= 0,02 mm
G_{xz} = 5,0 GPa	G_{xz} = 5,3 GPa	g_f = 18 kN/m	σ_{nc} = -60 MPa	σ_{nc} = -70 MPa
ν_{xy} = 0,30	ν_{xy} = 0,22		δ_{nc}= 1,125 mm	δ_{nc}= 0,065 mm
ν_{yz} = 0,52	ν_{yz} = 0,22			
ν_{xz} = 0,30	ν_{xz} = 0,21		<u>*shear direction*</u>	<u>*shear direction*</u>
σ_{xt} = 2200 MPa	σ_{xt} = 340 MPa		D_{22}= 53 GPa/m	D_{22}= 89 GPa/m
σ_{xc} = -1500 MPa	σ_{xc} = -163 MPa		τ_{xy} = +/- 12 MPa	τ_{xy} = +/- 5 MPa
σ_{yt} = 54 MPa	σ_{yt} = 340 MPa		δ_{xt} = +/- 0,225mm	δ_{xt} = +/- 0,056mm
σ_{yc} = -186 MPa	σ_{yc} = -163 MPa			
σ_{zt} = 54 MPa	σ_{zt} = 76 MPa			
σ_{zc} = -186 MPa	σ_{zc} = -76 MPa			
τ_{xy} = 85 MPa	τ_{xy} = 25 MPa			
τ_{yz} = 90 MPa	τ_{yz} = 25 MPa			
τ_{xz} = 120 MPa	τ_{xz} = 25 MPa			

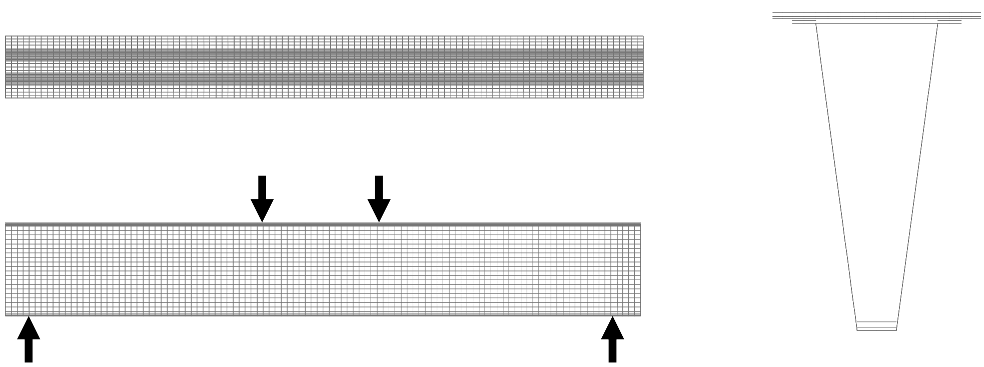


Figure 3. The mesh of the 3D FE model of the prototype beam, from above, a side view and the cross section.

5.2 Analyses

Non-linear FE analyses were conducted for both models. Comparison of the results from FE analyses of the two models reveals a reasonable conformity of behaviour between the models, especially concerning the composite parts of concrete and GFRP. The envisaged failure mode for the full-size beam was a local bond failure in shear within the bimaterial deck interface in the vicinity of the load, while for the test beam the expected failure was a combination of compression failure in the CRC concrete and a local bond failure close to the loading points. There was a possibility that the shear stresses in the webs also could become critical due to the decreased cross-sectional area of the webs, resulting from the down-scaling of the beam and the uncertainties in the material properties. However, there should be no risk of shear failure in the web if the adopted material properties were of the right magnitude.

Furthermore, the result from the analysis of the test beam showed acceptable agreement with the test results, thus demonstrating that the prototype beam behaved as expected. This is further discussed in section 7, where also some of the FE results are compared with the test results. Contour plots from the two beams' models where concrete were modelled with isotropic plasticity are presented in Figure 4 and Figure 5.

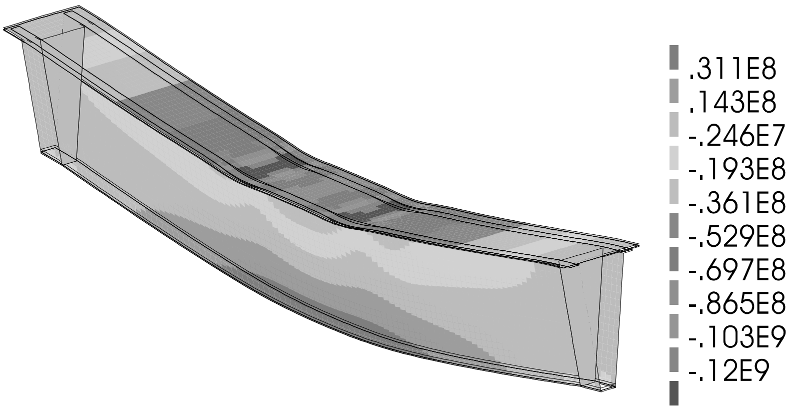


Figure 4. Contour plot of the deformed shape from FE analysis of the 5 m span prototype beam. Normal stresses [Pa] in longitudinal (x) direction at a load of 2 x 430 kN.

Figure 5. Contour plot of the deformed shape from FE analysis of the 25 m span full-scale bridge beam. Normal stresses [Pa] in longitudinal (x) direction at a central load of 900 kN.

Additionally, an analysis for control of Eigen values for buckling was conducted, stating that there would be no risk of buckling during the test. This was further verified by calculations according to Eurocomp (1996). Furthermore, from the FE analyses it was concluded that shrinkage would not constitute a problem since the maximum stress from shrinkage would be about 1 MPa. In addition, the shear stress from shrinkage in the bimaterial interface in the deck would act favourably, i.e. in the opposite direction, compared to the shear stresses from loading.

6. Laboratory test

6.1 Test performance

The beam was loaded in four-point bending. The set-up can be seen in Figure 6 and Figure 7. Strain transducers were mounted on both sides of the interface between the GFRP plate and the CRC, i.e. on the surface of the GFRP and facing down on short reinforcing bars cast into the CRC concrete. In addition, strain was also measured by means of strain transducers on the surface of the FRP and the CRC. Deflections and displacements were measured with LVDT gauges. Load cells accounted for the load measurement. The loading was deflection-controlled with a mid-span deflection rate of 0.5 mm per minute. This was decreased to 0.25 mm per minute when the load reached about 250 kN per jacking unit. In addition, the loading was interrupted continuously at predetermined loads to examine the beam.

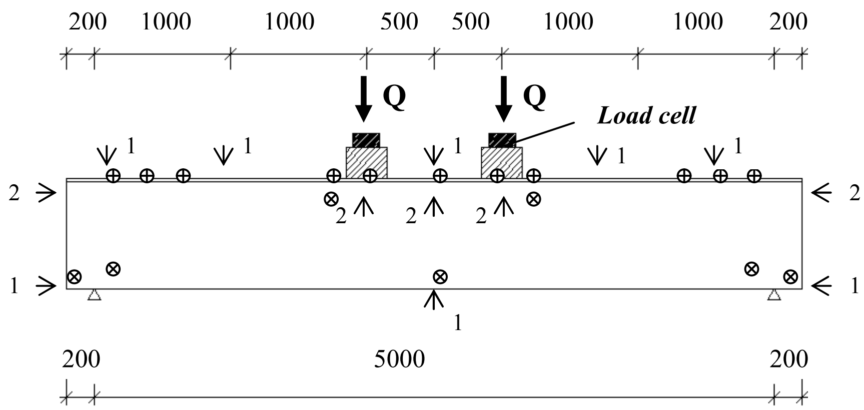


Figure 6. Set-up for the load test. Arrows denote LVDT transducers (number stands for one transducer in centre of beam and two transducers on corresponding sides respectively). Cross denote schematic placement of strain gauges (vertical cross for gauges in the deck and diagonal cross for gauges on the FRP beam).

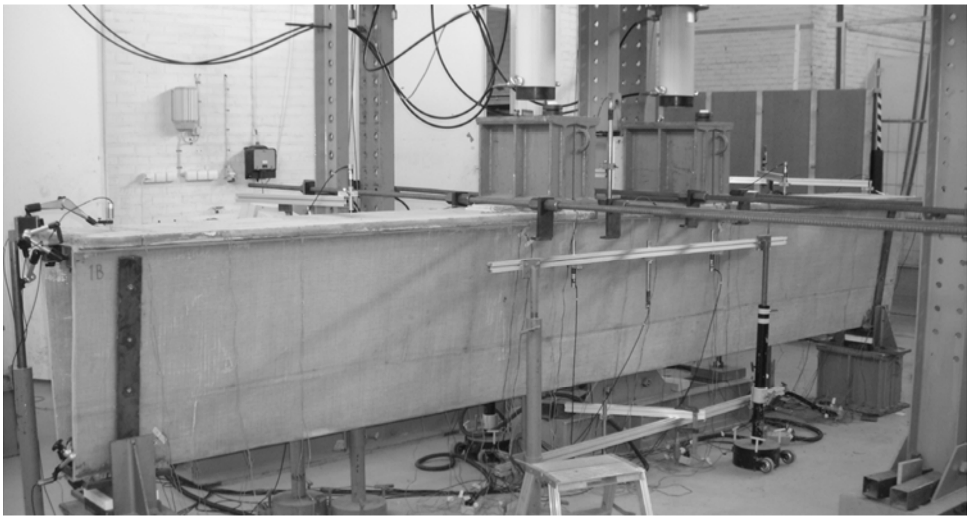


Figure 7. The prototype beam before loading.

6.2 Test results

As mentioned, the load test showed satisfactory agreement with the FE analyses. Thus, the structural performance predicted from the analyses could be verified. A comparison of the test results and the FE analyses is made in section 7, as noted earlier. Some of the results from the test are presented in Figure 8 and Figures 10–15. The presented results in the diagrams do not include effects of dead weight, etc., since the measurements are set to zero at the beginning of the loading in the evaluation.

The failure load reached was 2 x 429 kN and the failure was due to delamination in the GFRP plate in the deck, which was an unexpected failure mode. The failure was unfortunately induced by a transportation damage causing delamination of the plate, which was repaired prior to testing the beam. The defect can be noticed from Figure 8, showing the differential displacement between the bottom of the GFRP deck sheet and the centre of the CRC overlay at support 2. A significant displacement takes place on side A of the beam, which was the side that was damaged in the proximity of support 2 during transportation, while the displacement is practically zero on side B. In addition, no noteworthy differential displacements were measured at support 1. The beam after loading to failure is shown in Figure 9.

However, the expected failure, a combination of compression failure in the CRC and local bond failure in the bimaterial interface at the loading points in the deck, was just about to be reached when the beam failed, and the beam behaved in accordance with what was foreseen during the loading. For example, calculating the compression stress in the top of the CRC concrete at mid-span from the measured strain at failure (compare Figure 10), one obtains about 120 MPa in compression, hence demonstrating the vicinity of the ultimate compression stress. But in some respects, such as uneven distribution of strain and stresses over the cross-section, the damage is likely to have influenced the results, although it is not possible to judge how large this influence was. Nevertheless, in all essentials, apart from not reaching the expected failure mode, it is believed that the load test was successful and that the objectives of the test were achieved.

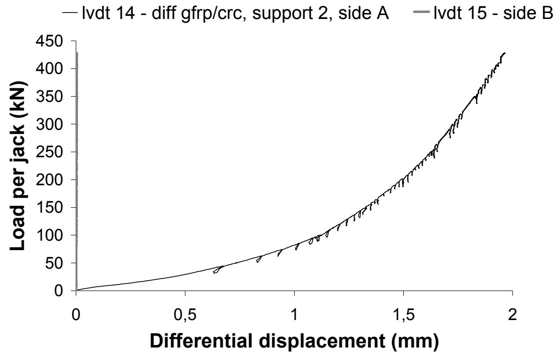


Figure 8. Differential displacement between the bottom of the GFRP deck sheet and the centre of the CRC overlay at support 2. Notice the significant displacement taking place on side A of the beam (the side which was damaged during transportation), while the displacement is practically zero on side B (the graph is almost parallel to the load axle).

Figure 9. The test beam after loading to failure.

Subsequent the failure the load dropped instantaneously to 2 x 258 kN (compare e.g. Figure 11), while the load was carried solely by the FRP beam without composite contribution from the deck. The thin part of the GFRP deck sheet, still connected to the beam (below the delamination), exhibited severe compression buckling.

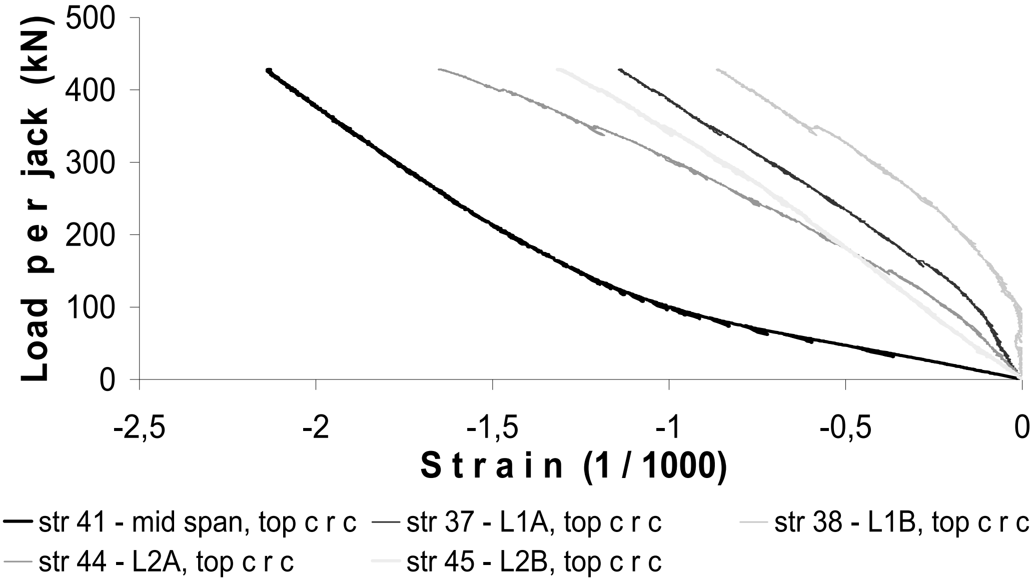


Figure 10. Strain measurements at mid-span and at the loading points for the top surface of the CRC. The mid-span strain was measured in the centre of the beam, while strains at the loading points were measured above the centre of the beam flanges on both sides just outside the load distribution device (towards the supports).

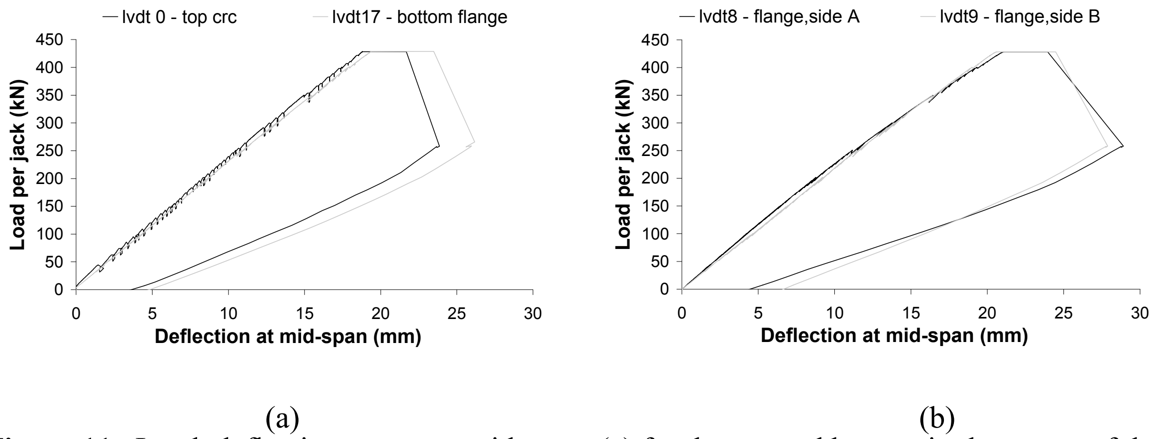
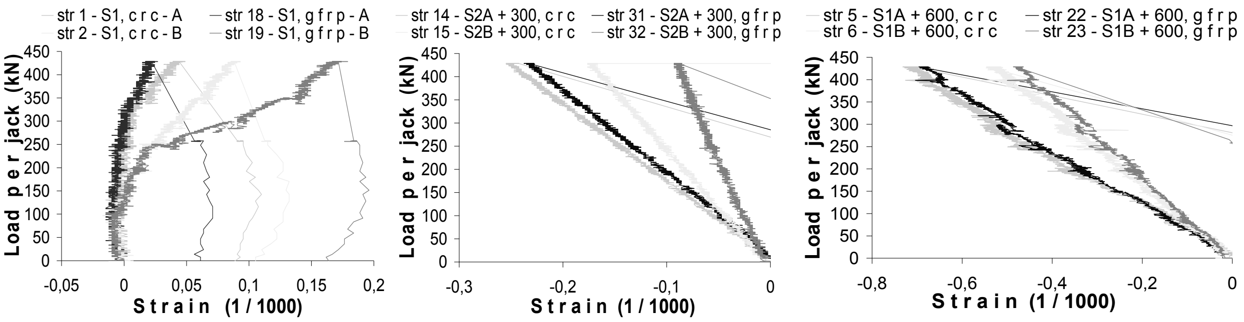
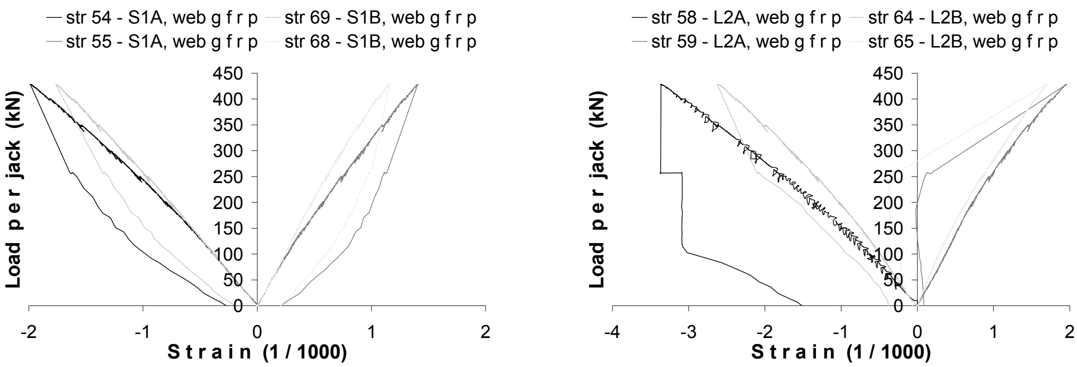


Figure 11. Load–deflection curves at mid-span: (a) for the top and bottom in the centre of the beam and (b) for the underside of the GFRP deck sheet at the edges.



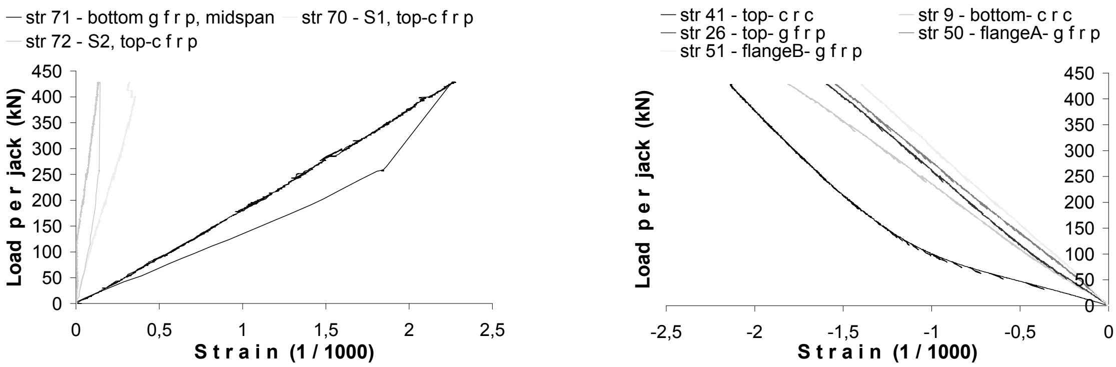
(a) (b) (c)

Figure 12. Strain measurements at the corresponding sides of the bimaterial interface in the deck for side A and side B of the beam (measured above the centre of the beam flanges) at various distances from the supports: (a) at support 1, (b) 300 mm from support 2 and (c) 600 mm from support 1.



(a) (b)

Figure 13. Strain measurements on the outer sides of the web surfaces in the two diagonal directions, at locations of maximum shear force: (a) at support 1 close to the bottom of the webs just outside the crossbeam, (b) at loading point 2 close to the top of the webs just outside the load distribution device.



(a) (b)

Figure 14. Strain measurements: (a) for bottom flange of GFRP at mid-span and top of CFRP on the protruding part just outside the crossbeam, and (b) at mid-span for top and bottom of CRC and top of GFRP deck sheet in the centre of the beam along with undersides of the GFRP deck sheet close to the edges.

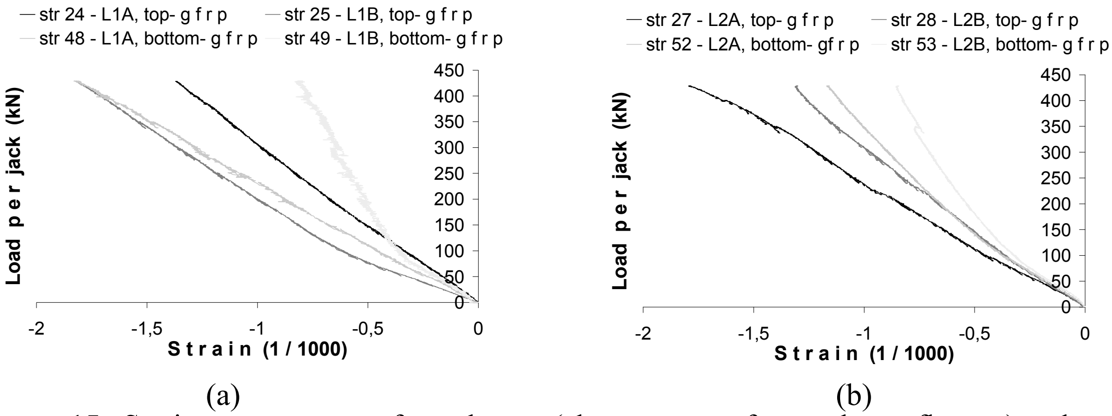


Figure 15. Strain measurements from the top (above centre of upper beam flanges) and bottom (just outside the edges of the beam flanges) of the GFRP deck sheet at loading points just outside the loading device (towards the supports) for beam sides A and B: (a) at loading point 1, (b) at loading point 2.

6.3 Materials testing of CRC

Tension tests

A series of six dog-bone-shaped specimens were cast for tension testing of the CRC concrete. The tests were done similarly to the experimental study; see Harryson (2008:II). The load, strain and displacement were measured with a load cell and through two strain gauges and two LVDT gauges mounted on the specimen.

A summary of the test results in terms of crack load and the corresponding calculated average tension stress, along with the failure load and the corresponding average calculated tension stress, is shown in Table 2. The crack load has been estimated from the test diagrams.

The failure loads were lower than expected, while the specimens did not show the strain hardening response that was supposed. The reason for this is probably the uneven distribution of steel fibres which was observed in the failure surfaces, which seems to be the result of too much vibration when casting the specimens. However, no uneven distribution of steel fibres was not noticed when checking cut-out specimens from concrete in the beam deck, where the distribution were found be satisfactory.

Table 2. Summary of results from tension test of CRC concrete.

Specimen	Crack load (kN)	Average Crack load (kN)	Calc. Tension stress (MPa)	Average calc. Tension stress (MPa)	Failure load (kN)	Average load at failure (kN)	Calc. Tension stress (MPa)	Average calc. Tension stress (MPa)
DB7	-		-		-		-	
DB8	12,2		8,1		12,2		8,1	
DB9	11,7	11,5	7,4	7,4	13,7	12,0	8,7	7,7
DB10	9,7		6,2		10,0		6,4	
DB11	12,2		7,8		12,2		7,8	
DB12	11,9		7,6		11,9		7,6	

__Compression tests__

One series from each of the three batches from casting, containing three cylinders each, was cast for compression tests and evaluation of elasticity modulus (on one series). The dimensions of the cylinders were ϕ100x200 mm^2. A summary of the results from the compression testing of CRC concrete and from the evaluation of the modulus of elasticity is presented in Table 3.

Table 3. Summary of results from the compression tests of CRC concrete.

Cylinder	Stress at failure f_{cc} (MPa)	Average f_{cc} (MPa)	Modulus of Elasticity E_c (GPa)	Average E_c (GPa)	Age (days)
1:1	153,4		58,0		
1:2	153,1	152,7	58,4	56,9	50
1:3	151,6		54,4		
2:1	-				
2:2	155,9	156,5			50
2:3	157,2				
3:1	155,6				
3:2	157,3	156,7			50
3:3	157,2				

7. Comparison of results and discussion

As has been mentioned, the load test exhibited reasonable agreement with the FE analyses. Thus, the structural performance predicted from the analyses was verified as can be concluded from the comparison of the test results and the FE analyses with isotropic plasticity of concrete, shown in Table 4 and Table 5.

Table 4. Comparison of deflections at failure load from the load test and the FE analyses. The failure load was 2 x 429 kN and the load in the FE analyses was 2 x 430 kN.

Placement	**Load Test**	**FE Analyses**	**Placement**	**Load Test**	**FE Analyses**
	Deflections (mm)	**Deflections (mm)**		**Deflections (mm)**	**Deflections (mm)**
__top-face CRC, centre of beam__			*__bottom-face GFRP-sheet__*		
mid-span (0)	18,8	18,7	mid-span-A (8)	21,1	18,8
1/5-point-S1 (5)	10,5	10,4	mid-span-B (9)	20,6	18,8
1/5-point-S2 (12)	10,6	10,4	L1A (6)	20,3	18,9
			L1B (7)	20,2	18,9
__bottom of beam__			L2A (10)	19,7	18,9
mid-sp.-gfrp (17)	20,2	18,6	L2B (11)	20,3	18,9

Notations: S – support, L – loading point, A –side A of beam, B – side B of beam, ( ) – the number of the LVDT gauges. The presented deflections are compensated for by settlements in the supports.

As seen in the tables, the scatter between the test and the analyses is sufficiently low compared to the uncertainties in the material parameters, especially for the GFRP, as has been mentioned. Hence, it can be stated that the adopted parameters are in the realistic range, e.g. the real moduli of elasticity should not differ much from those used in the FE analyses.

Table 5. Comparison of stresses and strains at failure load in the global longitudinal direction from the load test and the FE analyses. Load test stresses are calculated from the strains using the average measured elastic modulus for CRC and the elastic modulus assumed in the FE analyses for FRP. The failure load was 2 x 429 kN and the load in the FE analyses was 2 x 430 kN.

Interface in the deck	**Load Test**		**FE Analyses**		**Material faces**	**Load Test**		**FE Analyses**	
	Strain ε_x ($^o/_{oo}$)	**Stress** σ_x **(MPa)**	**Strain** ε_x ($^o/_{oo}$)	**Stress** σ_x **(MPa)**		**Strain** ε_x ($^o/_{oo}$)	**Stress** σ_x **(MPa)**	**Strain** ε_x ($^o/_{oo}$)	**Stress** σ_x **(MPa)**
300 mm from support, CRC					***top-face CRC***				
					L1A-crc (37)	-1,432	-81,4		
S1A-crc (3)	-0,220	-12,5			L1B-crc (38)	-0,868	-49,4	1,79	-98,7
S1B-crc (4)	-0,329	-18,7	-0,233	-12,8	L2A-crc (44)	-1,650	-93,3		
S2A-crc (14)	-0,254	-14,5			L2B-crc (45)	-1,322	-75,2		
S2B-crc (15)	-0,174	-9,9			mid-sp.-crc (41)	-2,134	-121,4	-1,31	-72,4
300 mm from support, GFRP					***bottom-face GFRP-sheet***				
S1A-gfrp (20)	-0,232	-4,9			L1A-gfrp (48)	-1,833	-38,7		
S1B-gfrp (21)	-0,264	-5,6	-0,231	-4,9	L1B-gfrp (49)	-0,828	-17,5	-0,976	-19,2
S2A-gfrp (31)	-0,235	-5,0			L2A-gfrp (52)	-1,165	-24,6		
S2B-gfrp (32)	-0,090	-1,9			L2A-gfrp (53)	-0,838	-17,7		
at load, CRC					***bottom-face GFRP-flange***				
L1A-crc (7)	-0.056	-3.2							
L1B-crc (8)	-1,541	-87,7			mid-span-A (50)	-1,541	-32,5		
L2A-crc (10)	-1,515	-86,2	-1,08	-58,3	mid-span-B (51)	-1,397	-29,5	-1,45	-30,2
L2B-crc (11)	-1,088	-61,9							
mid-span-crc(9)	-1,815	-103,3	-1,45	-79,7					
at load, GFRP					***bottom of beam***				
L1A-gfrp (24)	-1,363	-28,8			mid-sp.-gfrp (71)	2,274	48,0	2,22	47,2
L1B-gfrp (25)	-1,823	-38,5	-1,29	-25,7	S1-top-cfrp (70)	0,134	18,1	0,44	59,7
L2A-gfrp (27)	-1,797	-37,9			S2-top-cfrp (72)	0,320	43,2	0,44	59,7
L2B-gfrp (28)	-1,302	-27,5							
mid-sp.-gfrp(26)	-1,593	-33,6	-1,46	-30,7					

Notations: S – support, L – loading point, A – side A of beam, B – side B of beam, ( ) – the number of the strain transducers.

In all essentials, the prototype beam behaved linear elastic till failure, as expected. Since the test had to be terminated, followed by the sudden delamination in the GFRP deck sheet, it was not possible to continue and evaluate the residual strength of the FRP beam without composite action. From the analyses, this strength represents a load level of about 2 x 340 kN in pure bending, which is somewhat higher than the load 2 x 258 kN reached just after the failure.

It can be seen from the test results that the bimaterial interface in the deck is activated, but it is not possible to quantify the shear stresses in the interface from the measured difference in strain. However, for measurements at side A of support 1 a rough estimation from the differences in average strain between the measure points indicates a shear stress of 2 –3 MPa in the bimaterial interface, which seems reasonable. It can also be noticed that the distribution of stress seems to be uneven over the interface, probably resulting from the uneven response as a result of the deficiency in the GFRP deck sheet, as has been discussed. Furthermore, no signs of failure in the interface can be found, either from the test result or from the examination of the beam after testing. Even the interface at support 2 were undamaged despite the high energy release rate at the sudden interlaminar failure, compare Figure 9. Hence, the composite action seems to be intact.

In the transverse direction, the top of the CRC cracked at a load of about 2 x 40 kN beneath the loading points, and about 2 x 140 kN just outside the load distribution device, whereas the FE analyses predicted a crack load of about 2 x 130 kN. The proximity to the loading points is likely to have influenced the crack development severely, but the location of the cracks just inside the GFRP beam flanges was the same in both analyses and test.

As for the compression strains and stresses in the CRC, it seems likely that the results of the FE analyses in the mid-span section are somewhat unrealistic influenced by the loading arrangement, while at the loading points this is not the case. Hence, the agreement between analyses and test is better at the loading points than in mid-span. It is also possible that the small shrinkage cracks observed prior to testing somehow affected the results. Although not in coinciding points, the highest compressive stress in the CRC from the FE analyses was 137 MPa (for the load 2 x 430 kN) while the highest measured (calculated from the strain) was 121.4 MPa.

From the strain measurements in the diagonal directions in the web (see Figure 12), it can be seen that the stress levels are high, especially at the loading points. This demonstrates that the concern about the shear stress in the webs was justified, but also that the expected shear capacity was sufficient.

8. Conclusions

The prototype beam exhibited a satisfying structural behaviour during the test, and reasonable agreement with the predictions from the FE analyses. Despite the somewhat unexpected failure mode – delamination of the GFRP plate in the deck induced by a repaired damage – the beam behaved in accordance with what was foreseen during the loading. Thus, in all essentials the load test is believed to have been successful and, apart from not reaching the expected failure mode, the aims of the test were achieved. Since the test is unique, the results cannot be statistically verified, and further testing will be necessary in the forthcoming work before decisive recommendations about the performance of the beam can be made.

Acknowledgement

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