THESIS FOR THE DEGREE OF LICENTIATE OF ENGINEERING

Fibre reinforced polymer bridge decks: Sustainability and a novel panel-level connection

VALBONA MARA

Department of Civil and Environmental Engineering
Division of Structural Engineering, *Steel and Timber Structures*
CHALMERS UNIVERSITY OF TECHNOLOGY

Gothenburg, Sweden 2014

Fibre reinforced polymer bridge decks: Sustainability and a novel panel-level connection

VALBONA MARA

ISSN 1652-9146
Lic 2014:1
Department of Civil and Environmental Engineering
Division of Structural Engineering, Steel and Timber Structures
Chalmers University of Technology
SE-412 96 Gothenburg
Sweden
Telephone: + 46 (0)31-772 1000

Cover:
Experimental investigation of a novel panel-level connection for FRP bridge decks.

Chalmers reproservice
Gothenburg, Sweden 2014

Fibre reinforced polymer bridge decks: Sustainability and a novel panel-level connection

VALBONA MARA
Department Civil and Environmental Engineering
Division of Structural Engineering, Steel and Timber Structures
Chalmers University of Technology

ABSTRACT

Fibre reinforced polymer (FRP) bridge decks have emerged as a competitive alternative to traditional decking solutions for the refurbishment of existing bridges, as well as the construction of new ones, in the past two decades. FRP decks offer inherent properties such as light weight, high strength and high resistance to aggressive environments. In addition, the prefabrication of FRP decks brings the benefits of industrial bridge construction and rapid on-site assembly leading to the minimisation of traffic disturbance. Even though the use of FRP decks started in the early 1990s, the uptake of these decks has been slow in bridge construction and there remains a need for research in diverse technical areas to promote the widespread use of these decks.

The existing research and field applications of FRP decks were synthesised to recognize the standing level of knowledge and map out possible knowledge gaps. As an outcome several research needs were identified wherein two of them were: to determine the potential of bridges with FRP decks with respect to sustainability, and to develop connections which enable rapid on-site assembly. This thesis aims to contribute in bridging these knowledge gaps by investigating the sustainability of bridges with FRP decks and developing a novel panel-level connection for potential swift on-site assembly of FRP bridge deck panels.

The sustainability of bridges with FRP decks was evaluated using life-cycle cost (LCC) analysis and life-cycle assessment (LCA) with a focus on carbon emissions. An existing steel-concrete bridge with a deteriorated concrete deck was selected as a case study. Two scenarios were studied and analysed: the total replacement of the bridge and a bridge rehabilitation scenario in which the concrete deck is replaced by an FRP deck. The analyses revealed that the latter scenario contributes to potential cost savings over the life cycle of bridges in addition to a reduced environmental impact in terms of carbon emissions.

A novel panel-level connection was developed by following a process in which the client, designer, manufacturer and contractor were involved. Numerical analyses and experimental tests were conducted to investigate the overall structural behaviour and the load-carrying capacity of the developed panel-level connection. The results demonstrated that the connection exhibits sufficient load-carrying capacity and ductility, while the requirements in the serviceability limit state (SLS) were not fully satisfied due to geometric flaws in the connection modules. More experimental studies encompassing specimens with higher level of precision are therefore recommended to obtain enhanced performance in the serviceability limit state.

Key words: Bridge; Connection; FRP deck; Life-cycle cost; LCC; Life-cycle assessment; LCA; Sustainability

LIST OF PUBLICATIONS

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

I. Mara, V. and Haghani, R. (2014), "Review of FRP bridge decks: Structural and in-field performance", submitted to *Construction & Building Materials*

II. Mara, V., Haghani, R. and Harryson, P. (2014), "Bridge decks of fibre reinforced polymer (FRP): A sustainable solution", *Construction & Building Materials. 2014.* **50**(0): p. 190-199.

III. Mara, V., Haghani, R. and Al-Emrani, M. (2014), "A novel connection for fibre reinforced polymer bridge decks: Conceptual design and experimental investigation", to be submitted

AUTHOR'S CONTRIBUTIONS TO JOINTLY PUBLISHED PAPERS

Specification of the author's contribution to the appended papers is:

I. The author was responsible for a major part of the paper

II. The author was responsible for a major part of the paper

III. The author was responsible for a major part of the paper

ADDITIONAL PUBLICATIONS BY THE AUTHOR

- Mara, V., Al-Emrani, M., Kliger, R. " Upgrading of an Existing Concrete-Steel Bridge Using Fibre Reinforced Polymer Deck – A Feasibility Study" in *1st FRP Bridges Conference*, September 2012, London, England

- Mara, V. and Haghani, R. " Upgrading Bridges with Fibre Reinforced Polymer Decks – A Sustainable Solution" in *CECOM Conference*, November 2012, Krakow, Poland

- V. Mara, R. Haghani, A. Sagemo, L. Storck, and D. Nilsson "Comparative study of different bridge concepts based on life-cycle cost analyses and life-cycle assessment", in *Asia-Pacific Conference on FRP in Structures* (*APFIS 2013)*, December 2013, Melbourne, Australia

- D4.18: " Techniques of off-site fabrication and on-site assembly of new bridges", PANTURA, WP4, 2013

Contents

Preface

This thesis deals with the feasibility of fibre reinforced polymer (FRP) bridge decks in industrial bridge construction. The work was carried out at the Division of Structural Engineering, Steel and Timber Structures, Civil and Environmental Engineering Department, Chalmers University of Technology. The project is a part of the European project, PANTURA no. 265172, which is a research project that is co-financed by the European Commission FP7-ENV-2010 (Jan. 2011- Dec. 2013).

I am grateful to my supervisor, Associate Professor Mohammad Al-Emrani, and co-supervisor, Assistant Professor Reza Haghani for their advice and encouragement throughout. I would like to thank my examiner, Professor Robert Kliger, for his support and contribution to enable me finish this study. I am grateful to the technical staff at Chalmers, Lars Wahlström, for his valuable help in executing the experimental tests. I am also grateful to my colleagues for their presence and stimulating discussions.

Finally, I wish to thank my family, who always believed in my success, and especially my father, who encouraged and gave me endless faith to complete my studies.

Göteborg, 2014

Valbona Mara

1 Introduction

1.1 Background

The majority of existing bridges around the world are relatively old, suffering of various degradation problems such as corrosion, fatigue and exposure to environmental attacks, where the most prominent problem is deterioration of decks. Additionally, these bridges were constructed with substantially different demands on traffic loads and intensity and fail in many cases to meet the current needs for higher axle loads, larger traffic volumes and higher speed. Two major needs emerge from this situation:

- Many exiting bridges are and will be to a higher extent in the near future in need of strengthening and repair in order to allow for a safe continued use.

- In many cases, a total replacement of these bridges will be necessary in order to meet the increasing demands on larger transportation volume.

These needs become particularly challenging in densely populated areas, which is typical for many places in Europe. Bridge construction in large cities means dealing with a huge number of constraints and variables. For instance, today, traffic congestion is a well-known problem that transport administration authorities struggle with every day in densely populated cities. When the aspects of sustainability are considered, bridge construction processes must deal with social, environmental and economic impacts including traffic disruption, noise pollution, inefficient use of resources and increased user costs. All these aspects should be added to the usual technical aspects in bridge construction and rehabilitation, and their weight is becoming substantially higher, especially in urban areas.

Traditionally, the rehabilitation and construction of conventional short and medium span bridges involves cast in-place concrete. While this construction technique is suitable for construction of bridges in open construction zones with no or very limited traffic flow, they might not be a suitable choice for construction of bridges in urban areas. Cast in-situ concrete involves a massive use of manpower and substantial site activities for construction of formworks, placement of reinforcement and casting of the concrete. These extensive site activities increase the risk of accidents jeopardizing the safety of workers and road users, influence the mobility, increase air and noise pollution and generate a lot of waste on-site. In addition, these activities require a lot of space and time in the construction site, causing in return traffic disturbances, increased traffic management and road user costs and increased green-house gas emissions and energy use.

To overcome these problems, industrial concepts of accelerated bridge construction that use prefabricated bridge elements have started to attract increasing attention both in research and in field application in the past twenty years. The use of prefabricated bridge elements for construction of bridges offers several benefits such as:

- Reduced construction time.

- Improved constructability in constrained areas.

- Reduced adverse social impacts on the road users and the surrounding community.

- Safer working environment, improved user safety and rate of traffic accidents.

- Reduced user delay costs.

- Improved environmental impacts.

Prefabricated bridge elements can be made of different materials such as steel or concrete in a controlled environment off site and assembled in place at the bridge site. A widespread and mature example is the prefabrication of concrete decks. Other examples include prefabricated piers, abutments and combined steel girders and partial concrete decks can also be found. Installation of prefabricated elements is normally completed using standard cranes. It is also possible to fabricate larger portions of the bridge including complete superstructures, superstructures with integral piers, or even complete bridges and move them into place. A key issue to large-scale prefabrication is the methods used to move and install the elements. Typical crane installations are not normally used due to the weight of the systems.

Therefore, there exist several limitations of prefabricated or modular bridge elements with regard to transportation and construction equipment used for erection of the bridges. Transportation limits the prefabrication of bridge elements to certain sizes as well as to acceptable weights. The weight of prefabricated elements is also related to the capacity of construction equipment and cranes. In this regard, construction material selection plays a significant role. The use of light construction materials would facilitate the fabrication of larger modular bridge elements, which in turn would enhance on-site assembly of bridges. Hence, the need of incorporating light and durable materials in bridge construction is highlighted in order to ease and improve the bridge erection processes and further enhance the durability of bridges in order to deliver long-life bridges.

In this respect, fibre reinforced polymer (FRP) composite materials have paved the way to new opportunities in industrial bridge construction. The outstanding properties of composite materials, such as high stiffness and strength-to-weight ratio, high fatigue and corrosion resistance and potential weight saving benefits over conventional materials have made them suitable materials for use in structural bridge elements, particularly bridge decks. The primary reason of FRP materials being convenient for use in bridge decks is the characteristics of light weight and high strength in combination with resistance to fatigue and corrosion.

Research has been performed on FRP decks focusing mostly on stiffness and strength properties. Despite the research effort, there is still a lack of knowledge regarding how to design and construct bridges with FRP decks. To date, no design codes exist, a factor which has limited the widespread of the application of FRP decks. Another question that needs research in more detail is the connections between FRP deck panels and between the deck and the underlying (steel) girders in order to ensure fast on-site assembly. Practical and economically feasible solutions need to be developed in order to ensure structurally efficient connections and fast on-site assembly.

Furthermore, FRP decks might seem unattractive due to their higher initial cost compared with conventional decks. By considering only the initial costs, the advantages of FRP decks could easily be overlooked. In reality, there are costs beyond the initial costs that should be considered in the cost estimation of bridges. Life-cycle cost (LCC) analysis, which sums up the total life-cycle cost including all costs from acquisition to demolition, is a good evaluation method to assess the economic viability of bridges. In addition, the environmental impact of bridges in a life-cycle perspective has gained a lot of attention from bridge authorities due to today's extensive resource consumptions. Therefore, it is of interest to study the cost efficiency and environmental impact of bridges utilizing FRP decks compared to conventional bridge designs.

1.2 Aim and objectives

As the application of FRP decks becomes more widespread there is a need for bridging the gaps in knowledge about structural performance of such systems, technical requirements and practical issues. The overall aim of this project is to contribute in providing evidence on structural and economic feasibility of FRP deck systems and develop solutions in order to facilitate the application of such systems based on the technical needs. Within this overall aim, four main objectives are defined:

i. To review and evaluate the structural behaviour and in-service performance of FRP decks and define possible knowledge gaps for research.

ii. To study the performance of connections used at different levels of the bridges utilizing FRP decks as well as detailing of these connections. The detailing of connections is of particular interest for rapid, time-efficient assembly on site with the intention to prevent traffic disruption during the construction period.

iii. To investigate if the use of FRP bridge decks contributes to a more sustainable bridge construction focusing mainly on economic and environmental aspects.

iv. To develop and investigate from conceptual design to verification through testing an innovative panel-level connection for FRP decks which enable swift on-site assembly.

1.3 Methodology and approach

To realise the objectives of this thesis several methods have been used throughout. Firstly, the latest research and practice were synthesized and several experts and practitioners were consulted in order to characterize the structural behaviour and field performance of FRP decks as well as the performance of connection details between bridge elements. This led to the identification of needed further research and founded the motivation of the studies in this thesis.

To study if the use of FRP bridge decks contributes to a more sustainable bridge construction, a comparative analysis utilizing the approaches of life-cycle cost (LCC) analysis and life-cycle assessment (LCA) was conducted. An existing steel-concrete bridge with a deteriorated concrete deck was selected to serve as a case-study. Two scenarios were studied and analysed: the total replacement of the bridge and a bridge rehabilitation scenario in which the concrete deck was replaced with an FRP deck. Initially, the efficiency of FRP decks for upgrading the case-study bridge was examined through a finite element study. The LCC and LCA analyses were carried out using excel-based software developed by the author for the purpose of this study.

The development of an innovative panel-level connection for FRP decks started with a conceptual design. In the conceptual design, an approach where the client/customer, the designer, the manufacturer and the contractor were involved was followed. Afterwards, the finite element method was used to initially design and examine the overall structural behaviour of the connection. Experimental tests were performed to verify the design and study the performance and the load-carrying capacity of the connection. The experimental tests included only static bending tests.

1.4 Limitations

FRP decks can be applied on different bridge types consisting of steel, concrete or composite supporting systems. The scope of this report is limited to bridge applications of FRP decks on steel girders.

In the review mainly presented in Paper I, the focus was on the static performance of FRP decks; therefore the dynamic performance and the durability issues were not discussed.

Sustainability covers three main aspects being social, environmental and economic, which are mostly presented in Paper II. The social aspects are not discussed in detail but they are limited to the ones covered in the PANTURA project. In addition, the life-cycle assessment is limited to the evaluation of carbon emissions only.

In Paper III, only one specimen was tested and material testing of the connection module was not incorporated. Due to lack of material data and flaws in the connection modules, a direct comparison between the finite-element analysis and the conducted test could not be made.

1.5 Thesis outline

This thesis consists of an introductory section and three papers in which most of the study is presented.

Chapter 2 introduces the FRP decks, including constituent material and manufacturing methods, structural performance, design considerations and the most common applications of FRP decks for bridges. In addition, the effect of different core configurations for FRP decks to bending and shear stiffness based on finite element method is included.

Chapter 3 introduces the importance of life-cycle cost analysis and life-cycle assessment and presents a study of upgrading a bridge with an FRP deck in comparison with a traditional rehabilitation project.

Chapter 4 presents an innovative concept for panel-level connections for FRP decks. The development process of the connection from conceptual design to verification is described. The analyses of the connection by means of numerical modelling and experimental tests are reported.

In Chapter 5, the main conclusions from this work are drawn.

2 FRP bridge decks

2.1 Introduction

Fibre reinforced polymer (FRP) composite materials have been extensively used in military aviation and space applications since the 1960s, due to their low weight, high strength and significant durability advantages. Their application in civil engineering started in the late 1970s, for mainly strengthening purposes. Afterwards, the utilization of FRP materials for production of structural elements such as FRP reinforcing bars, FRP cables and tendons and FRP decks as well as girders for use in footbridges and road bridges was initiated.

The idea of using FRP materials for bridge decks was essentially a result of the technology transfer initiatives taken by the Federal Highway Administration (FHWA), U.S. Department of Transportation, in order to transfer and utilize the extensive knowledge base associated with military and space applications for bridge applications. These initiatives led to the foundation of a research project titled "Transfer of Composite Technology to Design and Construction of Bridges" in 1983 (Zureick et al. 1995). Since then, the research efforts have been directed toward developing fabrication methods to manufacture shapes and sections appropriate for bridge deck applications, understanding their behavior under simulated vehicular loads, and developing details and methods with which FRP decks can be designed and constructed.

2.2 Materials

Fibre reinforced polymer material is a combination of polymer resins, acting as a binder, with strong and stiff fibres which act as a reinforcement. Usually, fillers are also added to the resin to alter or enhance the material characteristics, such as to improve fire or ultraviolet (UV) resistance of the composite material. Typical fibre reinforcement materials used for civil engineering applications are glass, carbon, aramid or basalt fibres, while typical thermoset polymer resins are polyesters, vinyl esters and epoxies. The mechanical properties and the cost of these materials are given in Table 2.1. The price of various materials usually increases with increased mechanical properties. Carbon fibres have the highest stiffness compared to all the other fibres, but the price of carbon fibres is more than 50 times compared with glass fibres. Regarding the resin types, epoxy resins exhibit better structural and environmental resistance and they are comparatively more expensive.

In Table 2.1, the properties of different fibre-epoxy composites are tabulated as well. As it can be noted the properties of the composite materials are in between the fibre and resin properties, where the fibres contribute most to the stiffness and the strength of the composite material in the direction of the fibres, while the resin is used to transfer the loads to the fibres and protect the fibres. The difference in mechanical properties of dry fibres compared with the composite materials is illustrated in Figure 2.1.

Table 2.1 – Properties of fibres and resins used for civil engineering applications (Agarwal et al. 2006; Domone and Illston 2010)

Material	Tensile strength [MPa]	Tensile modulus of elasticity [GPa]	Glass-transition temperature T_g (°C)[1]	Cost (euros per kg)[2]
Fibres				
E-glass	2400	69	-	1.3
S-glass	3450	86	-	10
Carbon (high modulus)	5200	300	-	85-90
Carbon (high strain)	5020	260	-	52-55
Aramid (Kevlar 49)	2760	125	-	14
Basalt	2100	90	-	3.5-5
Resins				
Polyester	34.5-103.5	2-4.4	75-150	1
Vinyl ester	73-81	3.0-3.5	100	3.5-4
Epoxy	55-130	2.75-4.10	100-250	9
Epoxy-fibre composite (fibre weight fraction 65%)				
E-glass	760-1030	41	-	-
S2-glass	1690	52	-	-
Aramid 49	1150-1380	70-107	-	-
Carbon	2689-1930	130-172	-	-

[1] Glass-transition temperature is the temperature when the resins start to soften and lose the stiffness and strength.
[2] Reference Mostostal Warszawa S.A. (Poland), 2013

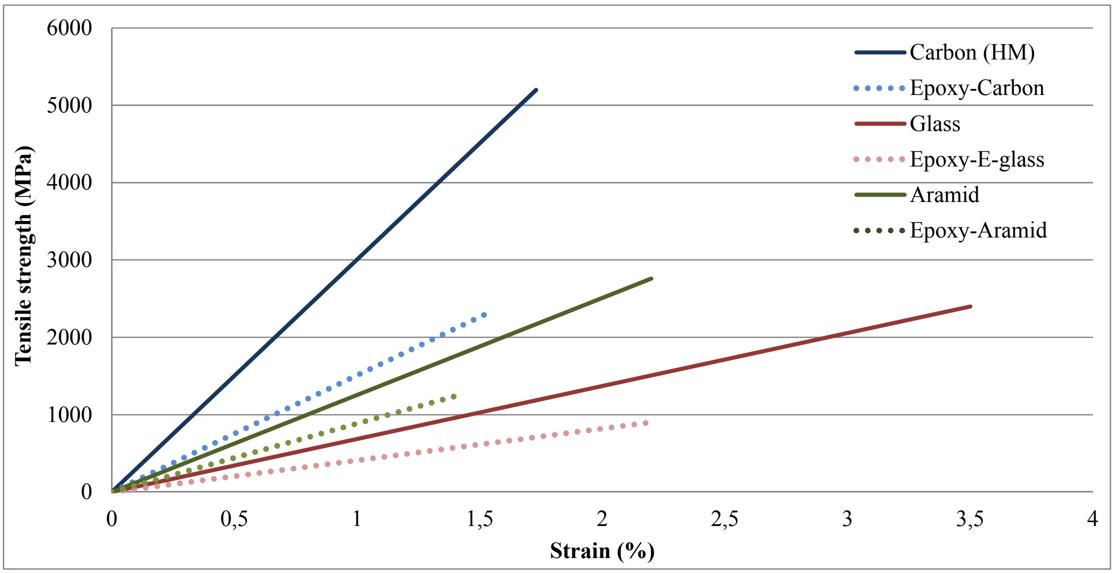


Figure 2.1 – Typical stress-strain relationship comparing the dry fibres with the epoxy-fibre composites

In addition to the mechanical properties, the degradation of composite materials over time is of great importance. As the resin protects the fibres from external influences, certain physical and chemical properties of the resins have a great influence on the durability of composites. Usually, it is the resin materials which control certain durability concerns for the composite materials. The fibres are generally quite durable. For instance, the fibres do not rust and they do not burn in case of fire. It is the resin material which is not durable to fire due to their glass transition temperature which is presented in Table 2.1.

In moisture and aqueous environments, apart from glass fibres which manifest some stiffness degradation, the fibres are not affected (Domone and Illston 2010). However, the stiffness and strength of the resins is degraded in moisture and aqueous solutions. Other factors which affect the stiffness and strength of the resins are temperature and ultraviolet radiation (Agarwal et al. 2006). Therefore, it is of great importance to select the appropriate resin for the composites in various applications.

2.3 FRP deck types

Fibre reinforced polymer decks usually consist of E-glass fibres and thermosetting polymer resins (polyester, vinyl ester or epoxy). Glass fibres are favoured for their substantially lower cost than other types of fibres (see Table 2.1). The fibres are in the form of roving, fabric or mats. The fabrics are made by aligning fibre strands together in multi-orientations mainly in 0^o, 90^o, or $\pm45^o$ (see Figure 2.2) and the mats can be produced by discontinuous or continuous fibres.

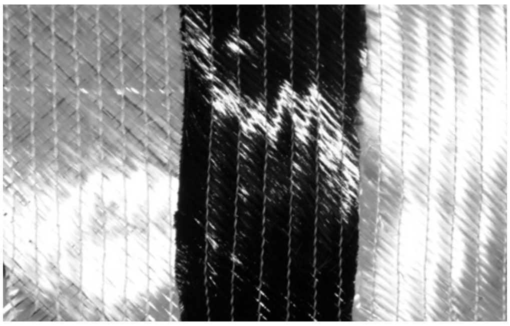
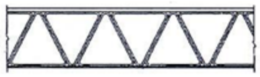
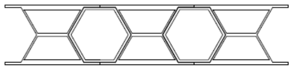
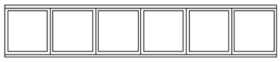
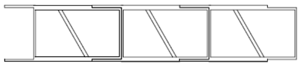
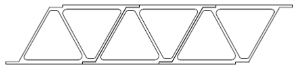
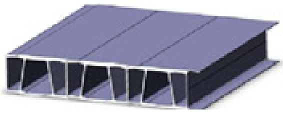


Figure 2.2 – Illustration of glass and carbon fabrics oriented in 45^o

FRP decks are discerned as pultruded decks or sandwich decks. Pultruded decks are produced by pultrusion manufacturing technique, while sandwich decks are manufactured by vacuum assisted resin transfer moulding (VARTM) or hand lay-up methods. Each manufacturing method has its own benefits. In general, pultrusion method produces good quality, consistent products with tight dimensional tolerances; however, prices tend to be slightly higher than for other methods. VARTM and hand lay-up methods allow the shape and dimensions of the sections to be changed along the composite member. More information on the manufacturing methods can be found on paper I.

Standard FRP decks are available on the market today, and the pultruded decks form the majority of them. An overview of the FRP deck systems available on the market today is depicted in Table 2.2.

Table 2.2 – Commercially available FRP deck systems

Deck system	Manufacturing process	Deck thickness (mm)	Deck weight (kN/m^2)	Manufacturer	Illustration of the deck
EZ-span deck	Pultrusion	216	0.96	Creative pultrusion Inc., USA	
Superdeck	Pultrusion	203	1.0	Creative pultrusion Inc. , USA	
Strongwell	Pultrusion	170	-	Strongwell, USA	
DuraSpan	Pultrusion	195	1.05	Martin Marietta Composites, USA	
ASSET	Pultrusion	225	0.93	Fiberline A/S, Denmark	
Delta deck	Pultrusion	200	-	Korea	
Hardcore deck	VARTM	variable	variable	Hardcore composites, USA	
Kansas deck	Hand lay-up	variable	variable	Kansas structural composites Inc. , USA	

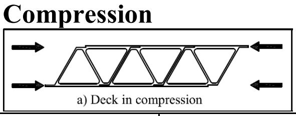
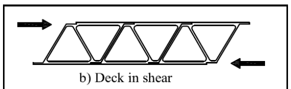
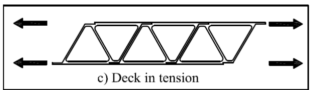
2.4 Structural performance of FRP decks

The mechanical properties and behaviour of FRP decks depend on several factors such as the constituent materials of the composite material, direction of the fibres, the method of manufacture, the cross-sectional geometry and the adhesives used for the deck component joints. In the design of FRP decks, the direction of fibres is optimized depending on the direction of loading. Fibres are usually aligned in 0/90^o to carry longitudinal and transverse loads respectively, and ±45^o to carry shear loads. The optimization of fibre orientations in the design of commercially available FRP decks is well done and modifications on the fibre directions would not result in extreme changes of the material properties. On the other hand, the cross-section configuration can have a great effect on the decks' properties and behaviour.

The unidirectional configuration of the pultruded decks gives the deck mainly a unidirectional load-carrying behaviour (in the direction of the pultrusion), whereas sandwich decks display a bi-directional load-carrying behaviour. The bi-directional behaviour is more favourable with regard to concentrated wheel loads. The concentrated wheel loads induce localized flexure of FRP decks, which need to be considered carefully in the design in order to avoid cracking of the wear surface.

The cross-section geometry of the cellular pultruded decks is varying from triangular to trapezoidal structures (see Table 2.2) which affects the properties of the decks. Decks with triangular configurations can carry higher shear loads and have higher shear stiffness than trapezoidal configurations in the transverse direction of pultrusion (see Table 2.3). The cellular nature of the decks influences the deck system properties In Table 2.3, the properties of the flanges of the decks are compared with the properties of the deck systems in compression, shear and tension.

Table 2.3 – Comparison of the properties of the flanges of the decks with the deck system properties (Coogler et al. 2005; Keller and Gürtler 2005, 2006; Keller and Schollmayer 2004)

	Compression 		**Shear** 		**Tension** 	
	Failure stress (MPa)	E-modulus (GPa)	Failure stress (MPa)	G-modulus (GPa)	Failure stress (MPa)	E-modulus (GPa)
Flanges	~-170	NA	~70	2.6~5.0	200~300	18~23
DuraSpan deck	-34	11.7	0.13	0.005	18	9.6
ASSET deck	-41	16.2	0.61	0.047	NA	NA

It is noticed that the properties of the deck systems are much lower than the properties of the flanges. For instance, the compression strength of the DuraSpan deck system is 20% of the material strength of the flange, which indicates that the material strength of the composite material is not fully exploited. This is attributed to the failure mode which starts at locations where through-thickness tensile stresses (peeling stresses) are exceeded due to high bending moments or axial tensile forces between the webs and the flanges of the decks. The failure is characterized as delamination failure of the webs from the flanges as shown in Figure 2.3.

Figure 2.3 – Failure of Asset deck subjected to in-plane compression (Keller and Gürtler 2006)

Composite materials are quite sensitive to peeling stresses and the strength of the material when subjected to pure peeling stresses was in one study (Keller and Vallée 2005) reported as low as 9.8 MPa, which is much lower than the material strength in the principle fibre directions. Therefore, special attention should be paid to loading situations that result in high tensile stresses through the thickness of the composite material.

Punching failure mode is also a common failure mode, which depends on the type of the patch load type. In experimental tests it is observed that punching failure occurs if steel plates are used for the patch load, while in case of simulated tire loads bending failure of the flanges takes place (Zhou et al. 2002). The different failure modes for pultruded decks in bending are illustrated in Figure 2.4. It should be noted that failures in the adhesives or in the adhesive interfaces usually do not occur.

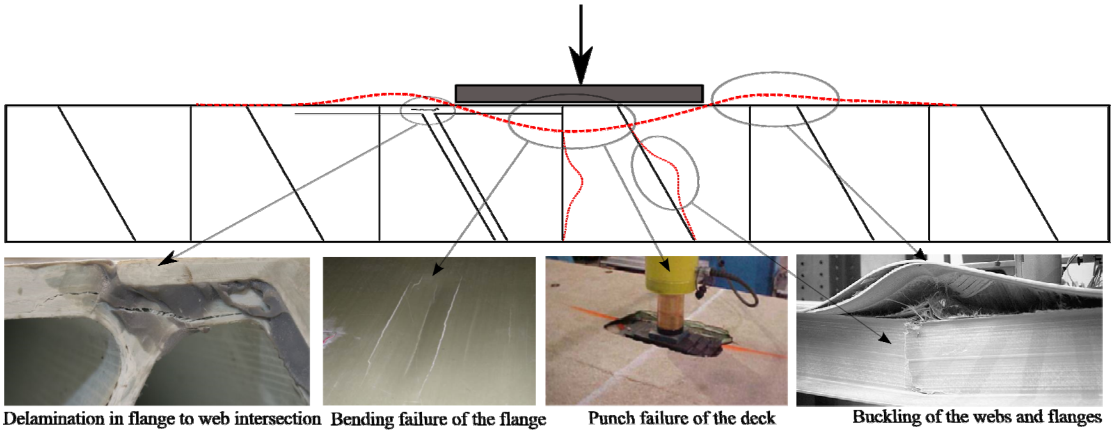


Figure 2.4 – Different failure modes of pultruded decks (Keller and Schollmayer 2004; Zhou et al. 2005)

The failure mode of sandwich decks under patch loading is characterized by debonding of the core from the flanges as shown in Figure 2.5. Hence, it is usually the shear strength of the adhesive which controls the strength of sandwich decks.

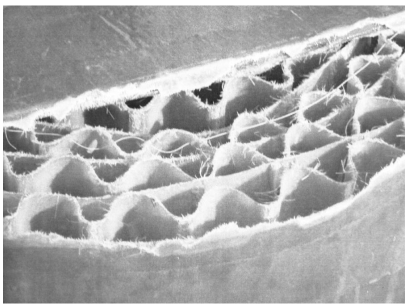


Figure 2.5 – Debonding of the flange from the core (Kalny et al. 2004)

The ultimate failure response of the FRP composite decks is usually linear elastic with slight non-linear envelope close to the failure loads. The ductile behaviour close to the failure load depends on the tested specimen sizes. If deck plates are tested rather than beams the ductility increases due to the plate action (Canning et al. 2007).

Regarding fatigue resistance of FRP decks and composite action between the deck and the supports, information can be retrieved in paper I.

2.5 FRP deck concepts

The majority of FRP bridge decks are pultruded decks and many studies have been conducted on the optimization of the cross-section configuration of the deck (Gan et al. 1999; Kim et al. 2005). The optimization of these decks has mainly been done without considering the required properties of the decks in case of composite action design between the deck and the supports. The composite action offers benefits of increased stiffness and strength of the systems as well as economic bridge designs.

In addition, FRP decks are usually developed to span over two supports as illustrated in Figure 2.6.

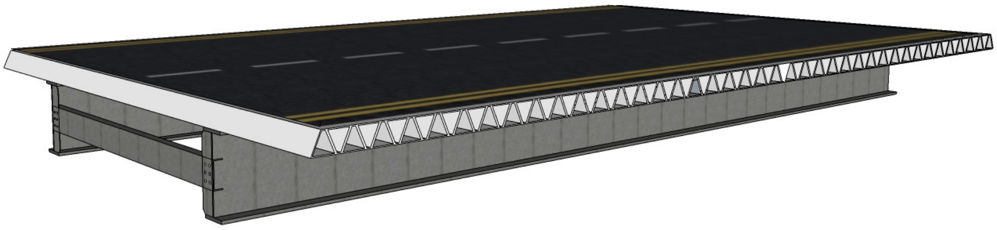


Figure 2.6 – Illustration of a typical bridge FRP-steel bridge where the deck is supported on the two steel girders

However, in other bridge concepts the decks can be supported on four or three edges. One example of such a bridge is shown in Figure 2.7 where the transverse beams in addition to stabilizing elements act as load-carrying members by supporting the deck.

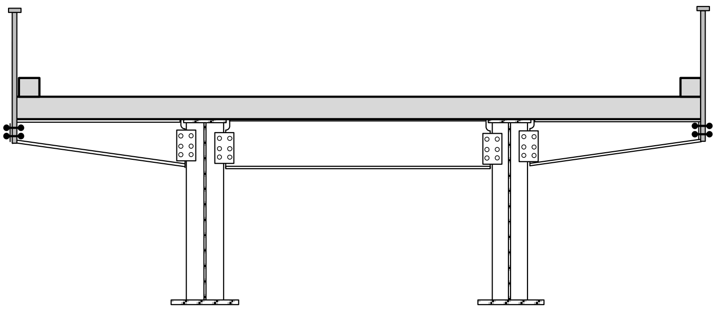


Figure 2.7 – Illustration of a bridge concept where the deck is supported on four or three edges

In these bridge cases, the decks are most effective if they offer a bi-directional plate bending behaviour. Motivated by this and the interest of utilizing the benefits of composite action, sandwich decks were selected to be studied. The effect of different core configurations on the properties of sandwich decks was examined.

Deck concepts with different core configurations were assessed using finite element analyses. The assessment covered the global bending stiffness and shear stiffness of the decks. The properties of the decks were compared and normalized with reference to the original design of the commercially available ASSET deck. The geometry of one pultruded element of the ASSET deck is specified in Figure 2.8.

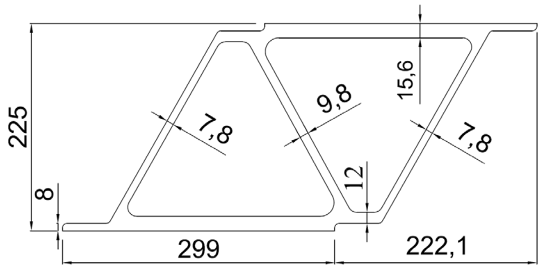


Figure 2.8 – Geometry of one component of the ASSET deck

The dimensions of the developed deck concepts were defined to ensure approximately the same volume of the material for each panel for the purpose of comparison. The geometric dimensions of each deck profiles are depicted in Table 2.4 and Figure 2.9.

Table 2.4 – Dimensions of the developed sandwich deck concepts

Height	220 mm
Thickness of the flanges	15 mm
Thickness of the webs	5 mm

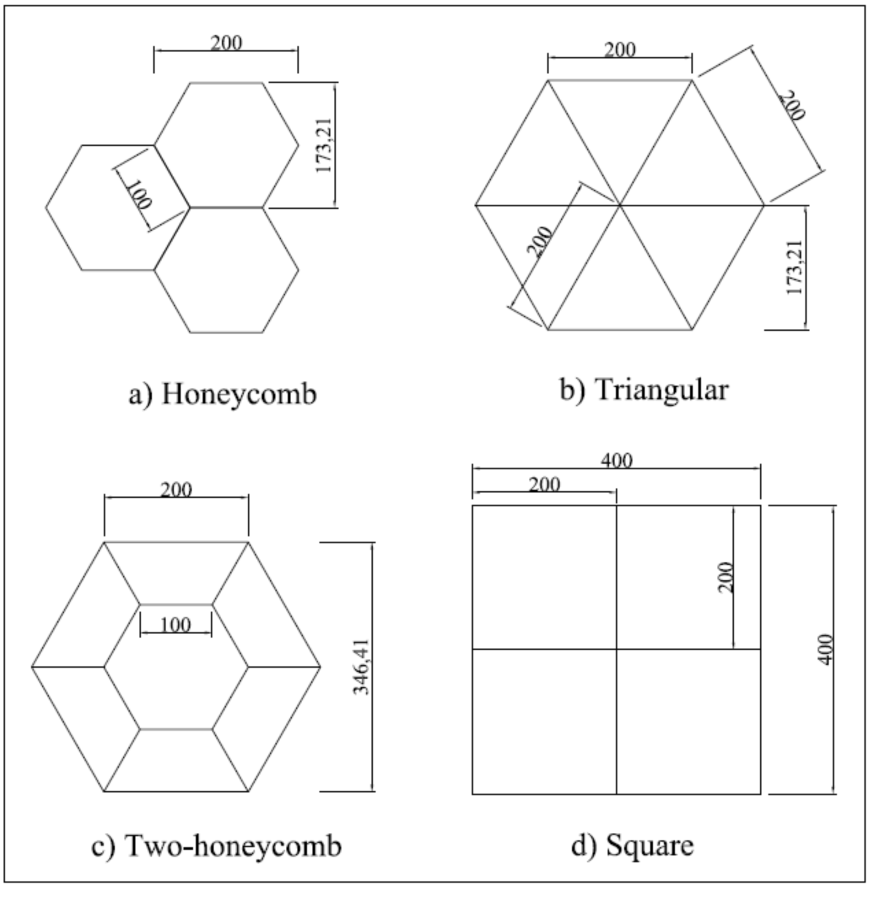


Figure 2.9 – Geometry and dimensions of sandwich deck core concepts (all dimensions in mm)

Finite element modelling:

Panels of dimensions 1 m wide and 3 m long were modelled in the available software ABAQUS (ver. 6.10-2). Uniformly distributed loads were applied instead of patch loads since the global performance was of interest in this preliminary study. In case of shear stiffness, uniform loads were applied per shear area. For the present purpose of comparison, the material was assumed to be isotropic in all decks and the results were normalized taking the ASSET deck as a reference. The loading and the support conditions for each load case are presented in Figure 2.10. The configurations of the reference ASSET deck and the other deck cores modelled in ABAQUS are shown in Figure 2.11.

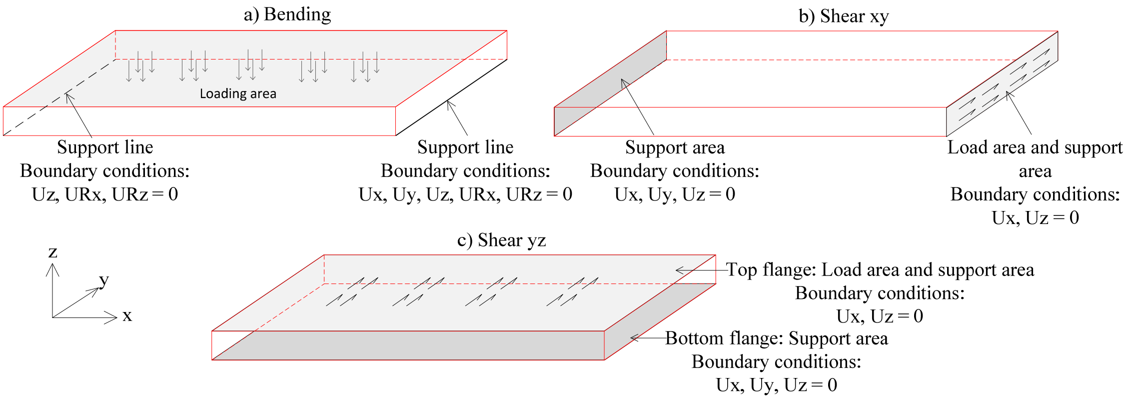


Figure 2.10 – Representation of loading and boundary conditions of the finite element models

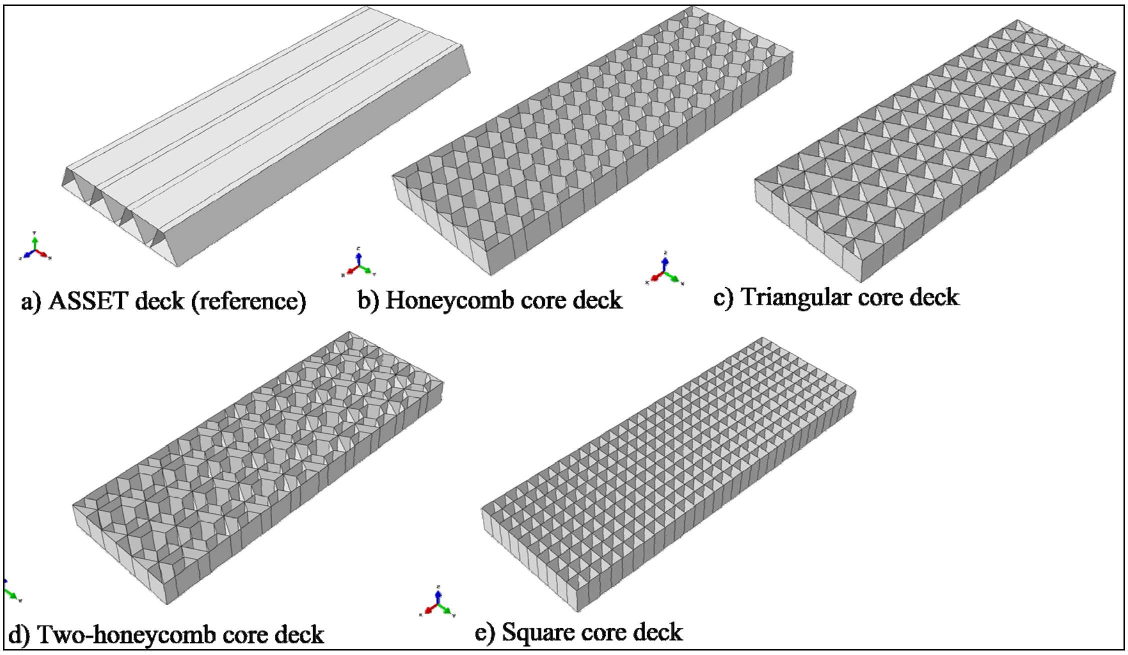


Figure 2.11 – Configuration of the ASSET deck and the sandwich deck cores modelled in ABAQUS

Results:

The deformations of the deck panels under the applied loads were measured and normalized in accordance with the ASSET deck. The results are shown in Table 2.5. The volume of each deck panel was not strictly the same as ASSET deck, but it was quite close with exception of the honeycomb deck.

The maximum deflections at the bottom of each panel were obtained and it can be seen that the global stiffness of all the decks is approximately the same, with exception of the honeycomb deck since less material is used. This is because the bending stiffness of the deck is primarily a function of the properties of the deck flanges and the depth of the deck, which were kept constant for all the decks in this study. It should be noted that the deck panels experience shear deflection as well, but its contribution is negligible.

Regarding the shear stiffness, G_{xy} and G_{yz} were compared with reference to the coordinate system given in Figure 2.10. The notification of the shear in this context is: the axis parallel to the applied shear force with the axis normal to that shear force.

Comparing the shear stiffness G_{yz}, which is important for the transfer of the strains from the bottom flange to the top flange in case of composite action, the square deck exhibits the highest stiffness and it is 10% higher than the shear stiffness of the ASSET deck. However, not the same advantage is observed for G_{xy} shear stiffness, which is needed to lower the shear lag effect if the deck will be working compositely with the supports. The triangular deck exhibits the highest G_{xy} shear stiffness among the sandwich decks, but it is 37% lower than the ASSET deck.

Table 2.5 – Results of the deck properties normalized according to the ASSET deck

Property	**Asset**	**Honeycomb**	**Triangular**	**Two-honeycomb**	**Square**
Volume	1	0,86	0,96	0,97	0,98
Bending deflection	1	1,14	1,08	1,09	1,05
Shear deflection xy	1	1,56	1,37	1,49	1,41
Shear deflection yz	1	1,49	1,01	1,02	0,91

These results reveal that the geometric shape of the core can play a role on the shear stiffness properties of the decks. The sandwich decks with triangular and square core shapes appear to be more advantageous in this study. Even though the G_{yz} shear stiffness of these decks compared to the ASSET deck are not significantly higher and in case of G_{xy} shear stiffness they are even lower, there is a difference between the ASSET deck and the other decks which cannot be exposed in these analyses. The ASSET deck is a unidirectional deck and distributes the loads in one way. The other decks are bidirectional decks which provide a load distribution in both ways and are more feasible when the decks are supported on four edges. This helps also in obtaining a smooth distribution of the stresses and the deflections under patch loads.

2.6 Design considerations

One of the difficulties associated with the design of bridges with FRP decks is the lack of design codes. There exist design guidelines for the design of FRP pedestrian bridge and road bridges((AASHTO) 2008; Agency et al. 2005), but there is no universally-accepted standard code for FRP decks for usage in road bridges. Most current designs of road bridges with FRP decks are carried out by following the specifications provided by the FRP deck manufacturers, which have been justified by proof tests. However, standard analysis and design procedures for FRP bridge decks are yet to be developed.

FRP composite bridge decks should meet the same design requirements as conventional bridge decks by following the contemporary approaches of structural design of load-bearing systems which are based on the limit state concepts. Overall, structural analysis is characterized to fulfil these limit states for different load cases defined in the Eurocodes. In the following, the considerations which have been taken in the design of bridges with FRP decks are summarised.

Serviceability Limit State (SLS)

The design of FRP decks is stiffness-driven and it is controlled by the deflection limits. The serviceability limit state is therefore the most important criterion in the design of FRP decks. There is no deflection limit suggested for FRP decks, but a limit ranging from L/300-L/800 is adopted in the design of various bridges with FRP decks (Alampalli and Kunin 2002; Alampalli et al. 2002), where L is the deck span. The limit L/800 is proposed in the AASHTO (American Association of State Highway and Transportation Officials) code and is derived for global bending of the entire superstructure of a bridge and not for FRP decks. The AASHTO code does not contain any mandatory requirements regarding the allowable live load deflection of decks spanning transversely between girders and nor does the Eurocode. The deflection limit should be introduced to prevent cracking of the wear surface, which is mostly related to the localized deflections of the FRP decks.

It is observed that even the design of the steel structure supporting the FRP deck is governed by serviceability limit state rather than ultimate limit state (Keller and Gürtler 2005) (D4.18, PANTURA). Typically, a deflection limit of L/800 of the supporting span length L has been used based on the AASHTO criteria for road bridges (Alampalli and Kunin 2002; Alampalli et al. 2002).

Another design criterion put forward for FRP decks in serviceability limit state is to keep anticipated strains under 20% of ultimate strength to avoid the risk of long-term creep rupture (Alampalli and Kunin 2002; Alampalli et al. 2002). This criterion is usually fulfilled because in the constructed bridges it is observed that the strains levels are usually lower than 20%.

Vibration is also another serviceability limit check that is critical for bridges with FRP materials due to the lightweight of the material. The bridges' natural frequencies should be kept outside a 'certain range', in order to limit undesirable amplifications of vibrations under pedestrian and vehicle traffic. Such 'certain range' is provided in the Eurocodes for dynamic analyses of road bridges, but experience from conventional bridges suggests that if the fundamental natural frequency falls above 5Hz, then the dynamic effects are not significant (Agency et al. 2005). Preliminary studies in the PANTURA project have shown that very large shifts in the natural frequencies occur when going from the unloaded to loaded configurations of the bridge. The reason is that the bridge's structural mass is in the same order of magnitude as the mass of the vehicles in transit. The first natural frequencies are less than 5 Hz when the vehicle masses are considered in the analyses. Such observations suggest that the dynamic behaviour of FRP-steel composite bridges should be investigated through refined dynamic analyses.

Ultimate Limit State (ULS)

In ultimate limit state, the essential requirement is to ensure adequate structural resistance and satisfy the expression:

$$E_d \leq R_d \,,$$

where E_d represents the load effect and R_d the design resistance.

This requirement is usually fulfilled since the deflection criterion governs the design. Failure to service load ratios are usually higher than 3, which indicate that the strength of material is not totally exploited.

Typically, FRP deck components are subjected to concurrently various stress effects (flexural, axial, shear) under different loading conditions. This implies that a combination of all stress components along with suitable composite failure criteria (e.g. Tsai-Wu failure theory) should be used in the analyses.

Fatigue Criteria

Fatigue design of a bridge with an FRP deck includes the check of the fatigue resistance of the deck material and connections in addition to other parts of the load-carrying system. It is suggested that at a minimum, the deck system should be capable of sustaining 2 million cycles at a load level of 1.5 times the wheel load (Karbhari 2001) without failure.

The connections in different levels should be capable of sustaining 2 million cycles under service loads with no change in structural response. The connections might be a source of stress concentrations in the FRP material when they are subjected to dynamic loading. Therefore, careful consideration should be given to local stresses due to local effects.

It is important to check if fatigue limit governs the design, because the ratio of live load to dead load stresses is higher in comparison to that in bridges with conventional decks. However, this ratio is low compared with the strength of the FRP material due to stiffness-driven design making FRP decks less vulnerable to fatigue, but it might be high for details as connections in the bridge design.

Other design requirements

Other design requirements which are considered in the design of bridges with FRP decks are:

- The exposed surfaces of the deck should be protected from ultraviolet exposure by using suitable paints. The ultraviolet component of sunlight degrades the polymers and therefore the composite material causing discoloration of the material and causing it to become brittle.
- Depending on the type of the connection of the FRP deck to the steel superstructures, some composite action between the deck and the steel is attained. However, in capacity calculations this composite action is not accounted for. Nevertheless, the connection itself should be designed to prevent failure since some load is inevitably transferred between the deck and the steel girders.
- Thorough analysis and quantification of the thermal effects in steel bridges with FRP decks should be undertaken. The FRP deck can heat up rapidly when exposed to direct sunlight, causing a thermal gradient between the top and the bottom surfaces of the FRP deck. The deck is not able to dissipate the heat from the top surface to the bottom as effectively as concrete decks. It is important to mention that thermal stresses may be as large as stresses resulting from live loads and this effect has been mainly observed in sandwich decks (Kong et al. 2013; Reising et al. 2004). In addition, it is important to account for the thermal stresses induced by the difference in thermal expansion coefficients of different materials, when for instance FRP deck is connected to steel girders.
- Acceptable wearing surfaces shall be designated from each particular manufacturer.

2.7 Applications of FRP decks for bridges

Numerous pedestrian and road bridges have been build or rehabilitated with FRP decks up to now, mostly in the United States, Korea and Europe. The focus in this thesis is given to road bridges. FRP decks have been implemented in various bridge projects, where most of them have been deck replacement projects, and they can be distinguished as follows:

1. Deck replacement projects for deteriorated decks or bridges with traffic load restrictions, where the reduction in dead load could benefit an increase in live load ratings (Alampalli and Kunin 2002; Alampalli et al. 2002).

2. Deck widening projects without imposing additional loads on the substructure.

3. Rehabilitation projects for historical bridges, avoiding the replacement of the bridge due to the cultural values (Alampalli and Kunin 2002; Fu et al. 2007; Grimm 2006).

4. Deck replacements for bascule bridges, benefiting from the lightweight of the deck to have simple mechanical systems for lifting or swinging and fast construction (Lewis et al. 2004; Sams 2005). One example in Europe is the Grasshopper bridge in Denmark, where the badly rotted timber deck whose planking had to be replaced about every five years was replaced with an FRP deck in 2011.

5. Superstructure replacement projects due to deterioration (Turner et al. 2004). One example is West Mill bridge, which is the first all-composite road bridges in Europe, and it was installed in 2002, in England (Canning 2012).

6. New bridges where accelerated construction is needed to reduce the cost of maintenance and protection of traffic and reduce traffic congestions (Knippers et al. 2010; Lee et al. 2010).

7. FRP decks have been commonly used on steel structures with spacing between girders smaller than 3 meters. The spans of the superstructures have been relatively short (<14m), but in some projects FRP decks are applied for larger spans (Knippers et al. 2010).

The in-service performance of the FRP decks in the abovementioned bridge applications is discussed in Paper I.

2.8 Summary and conclusions

Fibre reinforced polymer decks offer a number of advantages that can provide dynamic solutions for bridges as well as long-term benefits. Some of the most important advantages are listed below:

- **High specific strength and stiffness**: FRP composites exhibit high stiffness and strength-to-weight ratios compared to traditional materials. This characteristic makes FRP bridge decks to weigh approximately 20% of a structurally equivalent reinforced concrete deck. The weight savings in the superstructure translate to a decreased need for large foundations and ease of transportation and installation, leading to material and cost savings.
- **Corrosion resistance**: Unlike steel, composites do not rust, making these materials particularly attractive for bridge decks, where corrosion due to de-icing salts is of great concern for reinforced concrete or steel decks. Corrosion and its effects incur high maintenance costs in existing concrete and steel decks. These costs would be diminished by the use of FRP decks, leading to cost-effective bridge solutions.
- **Enhanced fatigue resistance**: Fibre reinforced composites are considered to be more resistant to fatigue than traditional materials. This is one of the major reasons for the application of composite materials in the aerospace industry. FRP decks have shown satisfactory results during fatigue tests, as described in Paper I. However, the fatigue resistance of connections should be considered in the design as it may control the life of bridges.
- **Tailored properties**: The attractiveness of FRP materials derives from the tailorability of fibre reinforcements. This feature makes possible to easily optimize the properties of the decks in order to fit specific requirements.
- **Rapid field installations**: Bridge construction is often characterized by long construction and installation periods, resulting in considerable inconvenience to users (e.g. traffic congestions, lane blockages, posting of speed limits, air and noise pollutions). In contrast, FRP decks or FRP deck-steel superstructures can be fabricated off site, shipped to the construction site and installed using light equipment. Hence the on-site work activities are

minimized. Off-site fabrication offers also the advantage of high-quality products and the potential for weather delays – which might be often the case with construction using conventional materials – is greatly reduced.

- **Increased safety**: Rapid installation of bridges incorporating FRP decks – owing to pre-fabrication – reduces the exposure time of workers and travelling public to onsite work activities, thereby, mitigating accidents and improving safety.

Although FRP bridge decks offer great opportunities, due to their relatively new practice in bridge constructions there remain still potential areas of investigation. The main areas which should be addressed based also on the outcome of Paper I include:

- Standardization of test methods, design practices and criteria and development of design codes.

- The relatively low stiffness of FRP decks leads to a deflection-driven design which does not allow the designer to fully take advantage of the materials strength. Development of decks which would balance the stiffness with the strength requirements is recommended. In addition, these decks should provide a more ductile behaviour prior to failure.

- Development of appropriate connections in panel and system level for FRP decks as well as the anchorage of bridge railings.

- Some bridges in service have experienced cracking or debonding of the wear surface. The most common used wear surface has been polymer concrete due to its enhanced durability and lightweight, but asphalt has been utilized as well. Development of appropriate wear surfaces for application to FRP decks is required.

- The high initial cost of FRP decks compared with an equivalent reinforced concrete deck makes bridge authorities reluctant to select FRP decks as an option. Therefore, comparative studies of life-cycle cost as well as life-cycle assessment for bridges with FRP decks are necessary.

- The oldest road bridge utilizing FRP deck dates back to 1996, No-name Creek Bridge in Kansas, US (Zhou et al. 2007). Hence, the in-service performance of FRP decks is known for less than 20 years. Long-term durability data are required to strengthen the claims that FRP decks have enhanced durability properties.

3 Bridges with FRP decks from a sustainability point of view

3.1 Introduction

Today, there is an increasing demand to make building structures more ‘sustainable’. The most widely accepted definition of sustainability, part of the concept sustainable development, is that of the Brundtland Commission of the United Nations, in 1987: “sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs.” Sustainability embraces three main aspects: economical, environmental and social sustainability, which are described more in detail in Paper II.

While traditional design and construction is focused on performance, cost and quality principles, sustainable design and construction adds to these principles the principle of minimizing the use of resources and the impact on the environment at all stages of use from initial construction to eventual removal and replacement. The construction sector is one of the largest industrial sector regarding resource consumption (Gervásio and da Silva 2008), which has led to an increased concern for sustainable development in order to keep the economy and environment in balance.

Due to short-sight thinking when making investment decisions, unnecessary costs and environmental impacts are contributing factors to the high resource consumption. The bridge authorities are therefore requiring to incorporate sustainable principles in the design of bridges. The most important requirements, which a questionnaire in the EU project PANTURA (Haghani 2011) revealed, specified by bridge authorities for maintenance and new bridge construction activities are given in Table 3.1. It was noticed that these demands belong to the sustainable principles and therefore are presented in Table 3.1 as a categorization of the sustainability pillars, being social, environmental and economic sustainability.

Table 3.1 - Client requirements for bridge construction w.r.t. sustainability pillars

Social sustainability	**Environmental sustainability**	**Economic sustainability**
• Short construction time • Minimize noise and traffic disruption	• Minimize carbon emissions • Minimize total use of materials • Minimize waste production • Possibilities to reuse and recycle the materials	• Reduce initial costs • Reduce maintenance costs • Reduce life cycle costs

3.2 Life-cycle approaches

3.2.1 Introduction

A life-cycle and holistic approach is very important with respect to design and construction of bridges, because the life-cycle of a bridge involves more than just the initial construction. Apart from the initial construction, maintenance, repair and disposal of the bridge can be quite costly activities and consume matter and energy. The main phases of the life cycle of a bridge are presented in Figure 3.1.

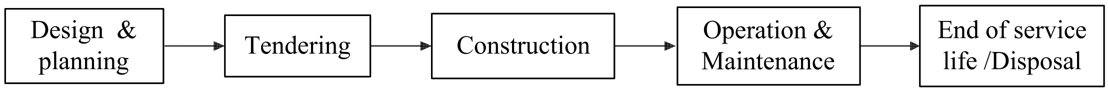


Figure 3.1 - Life-cycle stages of a bridge

Life-cycle modelling is an important decision-making tool in the tendering phase for choosing the most promising bridge design under certain conditions. If a life-cycle approach is followed, changes are easier to make in the design phase and the costs of changes will be lower.

Life-cycle approaches, such as life-cycle assessment (LCA) and life cycle-cost (LCC) analyses are developed in order to account for the environmental impact and total costs of a bridge design throughout its life span. These approaches are described in the following.

3.2.2 Life-cycle cost analysis (LCC)

Life-cycle cost analysis is an efficient tool to estimate the total costs of a bridge throughout its life span. Total life-cycle costs comprise different cost entities during the entire life of a bridge, as presented in Figure 3.2.

Principally life-cycle costs are divided into agency costs and social costs. Agency costs, including initial construction costs, operation and maintenance costs and disposal costs, mainly consist of the same cost units of material, transportation, equipment and labour costs. One challenge with operation and maintenance costs is the determination of the maintenance timelines and activities, including inspection, repairs and upgrading. This becomes even more delicate with new technologies due to the lack of data for maintenance activities.

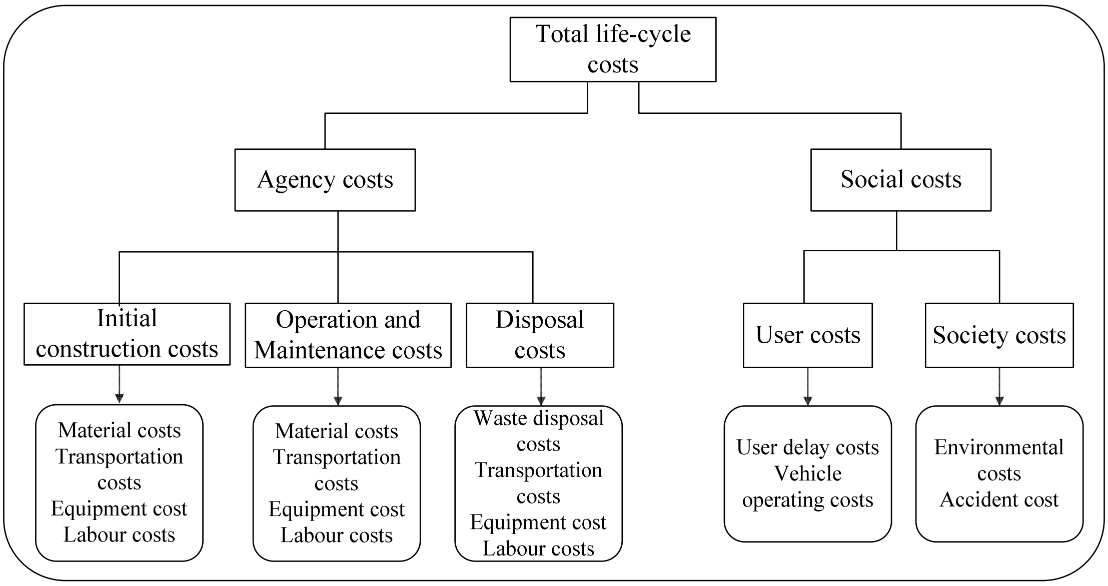


Figure 3.2 - Life-cycle cost analysis model

Disposal costs are related to the costs in the end of life of the bridge. Sometimes, waste disposal costs can be a profit if the waste materials are brought to recycling plants instead of landfills. For instance, recycling of steel material is very well practiced but concrete is usually disposed to landfill.

Social costs are mainly related to the traffic and society and are divided into user costs and society costs. User costs include user delay costs and vehicle operating costs and can be calculated with equations (3.1) and (3.2). Society costs consider mainly the environmental costs and accident costs. The accident costs during construction are computed by the provided equation (3.3).

$$User\ delay\ \cos ts = \left(\frac{L}{S_a} - \frac{L}{S_n}\right) \times ADT \times N \times w \qquad (3.1)$$

$$Vehicle\ operating\ \cos ts = \left(\frac{L}{S_a} - \frac{L}{S_n}\right) \times ADT \times N \times r \qquad (3.2)$$

$$Accident\ \cos ts = L \times ADT \times N \times (A_a - A_n) \times c_a \qquad (3.3)$$

where, L is length of affected roadway the cars are driving, S_a is traffic speed during bridge work activity, S_n is normal traffic speed, ADT is average daily traffic, N is number of days of roadwork, w is hourly time value of drivers, r is hourly vehicle operating cost, A_a is accident rate during construction, A_n is normal construction rate and c_a is cost per accident.

The parameter $\left(\frac{L}{S_a} - \frac{L}{S_n}\right)$ yields the time loss (additional driving time compared to normal conditions) of road users during construction activities. It is noted that user costs depend directly on this time loss, average daily traffic and number of days of roadwork. Thus, if construction can be performed as fast as possible, the user costs are minimized.

Environmental costs account for the damage of the pollutants to the environment. For instance, the total amount of carbon emissions can be converted to costs and added to the total life-cycle costs.

Once all the costs are calculated, they should be converted to the present value to be able to account for the loss of the money value. The costs are converted to the present value by considering a discount rate as in the equation (3.4):

$$C_{today} = \frac{C_{future}}{(1+d)^i} \qquad (3.4)$$

where, *d* represents the discount rate and *i* represents the time period usually in years

3.2.3 Life-cycle assessment (LCA)

Life-cycle assessment (LCA) is a framework to quantify the environmental impacts of a product throughout its lifecycle including: material acquisition/fabrication, use and final disposal or recycling. In recent years, the interest in LCA has grown in the construction industry (Du 2012; Hammervold et al. 2009) and its practical use needs to be further evaluated.

The standardized LCA is divided into four phases: i) goal and scope definition, ii) inventory analysis, iii) impact assessment, and iv) interpretation (see Figure 3.3).

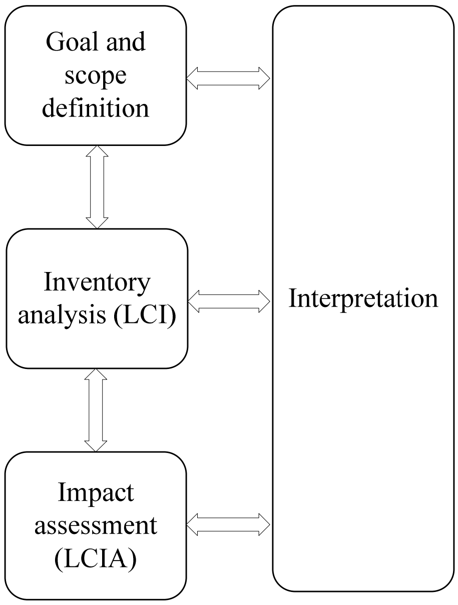


Figure 3.3 - The four phases of life-cycle assessment (LCA) as defined in ISO14040 (ISO 2006)

In the first phase, the goal and scope of the LCA are defined. The scope should include the function of the studied unit (the functional unit), the system boundary, the selected environmental impact categories and methodologies, limitations, and assumptions. The system boundary defines which life-cycle stages, inputs and outputs are included in the LCA. There are two different approaches on which life-cycle stages should be included in an LCA analysis: the cradle-to-grave approach and the cradle-to-gate approach (Zimoch and Rius 2012). Cradle-to-grave means that all stages from raw material acquisition to disposal are included, whereas cradle-to-gate considers all steps from raw material acquisition until the product is ready to leave the gate of the manufacturer.

Life-cycle inventory (LCI) is the second phase of the LCA and includes data collection and quantification. Material and energy input, and output in terms of emissions should be defined. Databases can be used for data collections from reports or softwares as such as Ecoinvent or Ecobalance LCA database (Du 2012; Hammervold et al. 2009; Lippiatt 2009). The collected data can then be used to quantify the flows of emissions, materials and energy. It is important here to choose data that corresponds to the studied situation since emission and material data can differ largely for two similar products due to, for instance, differences in production technique (Du 2012).

In the third phase, the life-cycle impact assessment (LCIA), the results of the LCI are assessed to find the environmental impact of the evaluated system. There are several different LCIA methods, among others ReCiPe, EDIP, Stepwise and Impact2002+ (GreenDelta GmbH 2013). A set of environmental impact categories is chosen and each flow quantified in the LCI is then connected to its related impact categories (Gervásio and da Silva 2008; Gervásio and da Silva 2013).

The LCIA can be divided into six elements, three mandatory and three optional. The mandatory elements are: selection of impact categories, category indicators and characterization models, classification and characterization while the optional are normalization, grouping and weighting. An example is depicted in Figure 3.4, where the selected impact categories are: acidification potential (AP), eutrophication potential (EP), global warming potential (GWP), abiotic depletion potential (ADP), ozone depletion potential (ODP) and photochemical ozone creation potential (POCP). Firstly, the emissions resulting in the inventory analyses are classified to the relevant environmental impacts. For example, flows of greenhouse gases are connected to the impact category global warming potential (Lippiatt 2009) and the flow of SO_2 is connected to acidification potential. In the characterization phase all the relevant flows for an impact are transferred into a common unit. For instance, all greenhouse gases that are connected to the global warming potential, such as carbon dioxide, methane, and ozone, will be expressed in one common unit, CO_2-equivalents (Du 2012; Lippiatt 2009). Normalization is a process through which the impacts of an alternative are put in relation to a reference value for the entire impact of a region, country or per person (Baumann and Tillman 2004; Gervásio and da Silva 2008; Hammervold et al. 2009). In the final weighting step, the different environmental impacts are rated by their importance for the overall environmental performance, which aims to make the results of LCAs comparable for different alternatives.

The final phase of LCA is the interpretation of results, where the results of the previous phases are evaluated in relation to the goal and scope stated in the beginning of the process. The interpreted results can be used as support in the decision-making process, keeping in mind though that the LCA is based on estimations and not actual numbers (Hammervold et al. 2009; ISO 2006), being aware of the uncertainties and limitations of the LCA study.

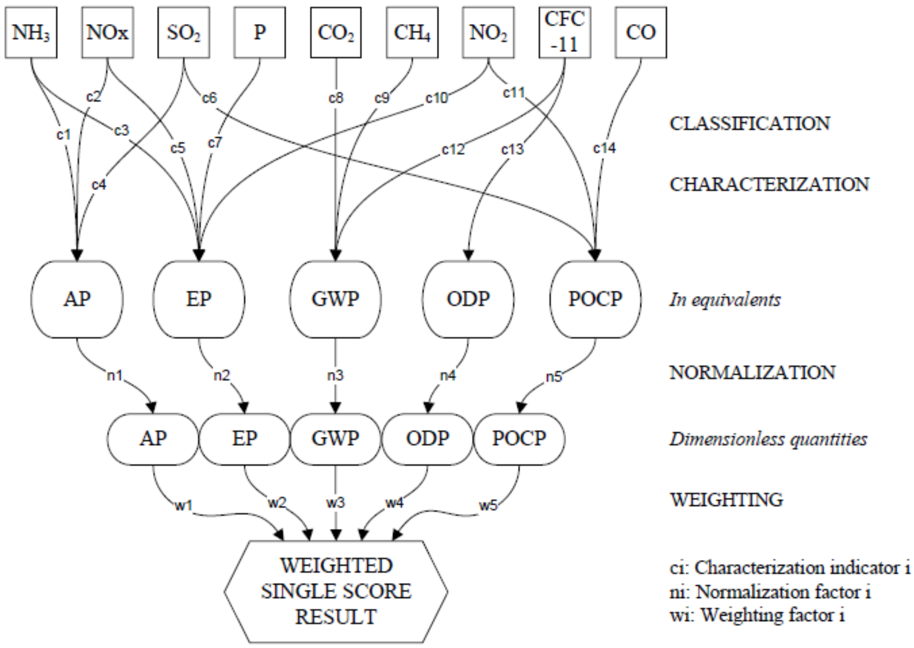


Figure 3.4 – An example of an LCIA (Hammervold et al. 2009)

In this study, the life-cycle assessment is limited to the life-cycle inventory phase by focusing on the carbon emissions which is the greenhouse gas that contributes most to the global warming potential.

3.3 Case-study bridge – LCC & LCA analysis

3.3.1 Introduction

The contribution of FRP decks to sustainable bridges is not well-known. Hence, in order to reveal the sustainable viability of FRP decks, a comparative study in a typical bridge refurbishment project is performed. Information on the case study bridge is given in Paper II. The study includes life-cycle cost analysis and life-cycle assessment in terms of carbon emissions. Two different options are considered for the refurbishment:

1) Replacement of the entire existing superstructure with a prefabricated concrete deck on steel girders

2) Replacement of the concrete deck with an FRP deck. (Note: replacement of the existing concrete deck with a new concrete deck was not an option due to limited load-carrying capacity of the steel girders)

The structural analyses of the option of replacing the concrete deck with an FRP deck were performed by means of the finite element method and can be found in (Mara et al. 2012).

3.3.2 Life-cycle cost analysis

The costs are tallied for each life-cycle stage of the bridge, converted to present-day money value, and compared between the two alternatives. In order to define the costs for each stage, a life-cycle inventory is done for both cases. These are presented in the diagram flows in Figure 3.5 and Figure 3.6.

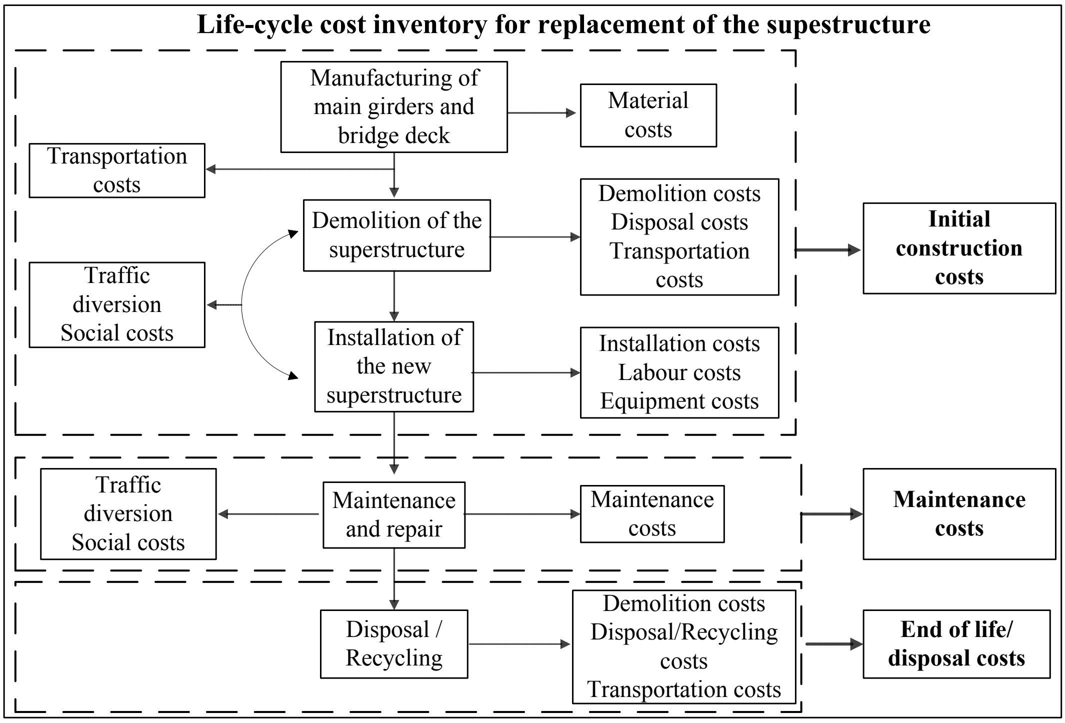


Figure 3.5 – Life-cycle cost inventory for replacement of the superstructure

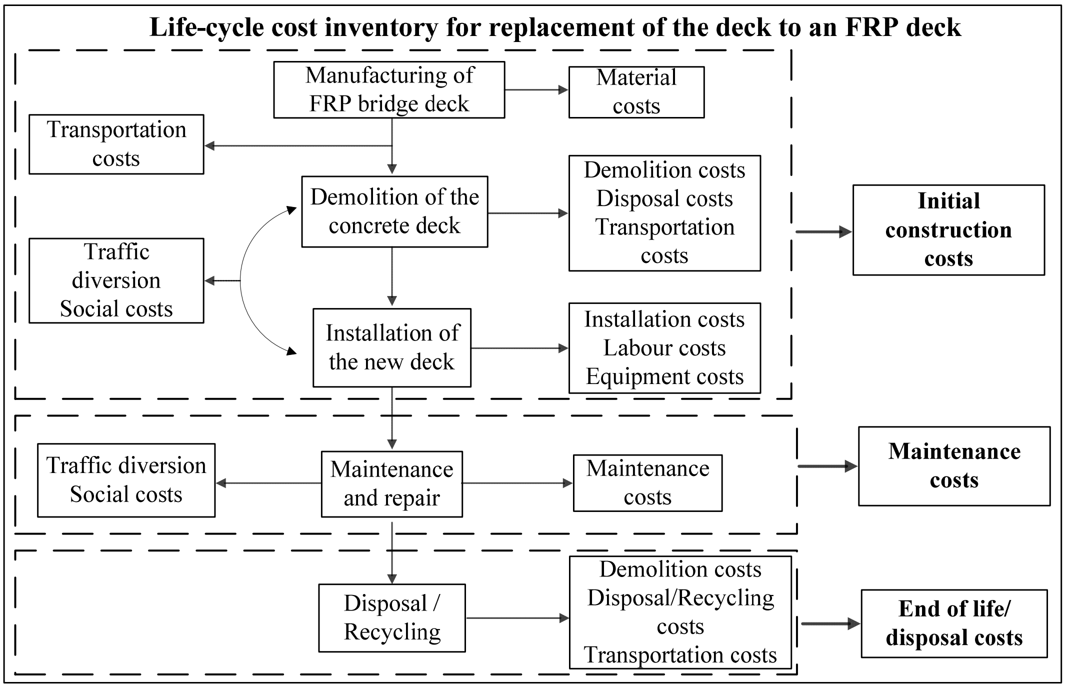


Figure 3.6 – Life-cycle cost inventory for replacement of the deck to an FRP deck

The considered maintenance activities in this study during the life cycle of the bridges for both alternatives are summarized in detail in Paper II.

The result of the total costs over the intended service life of 80 years is presented in Figure 3.7. A breakdown of all these costs is presented in Paper II.

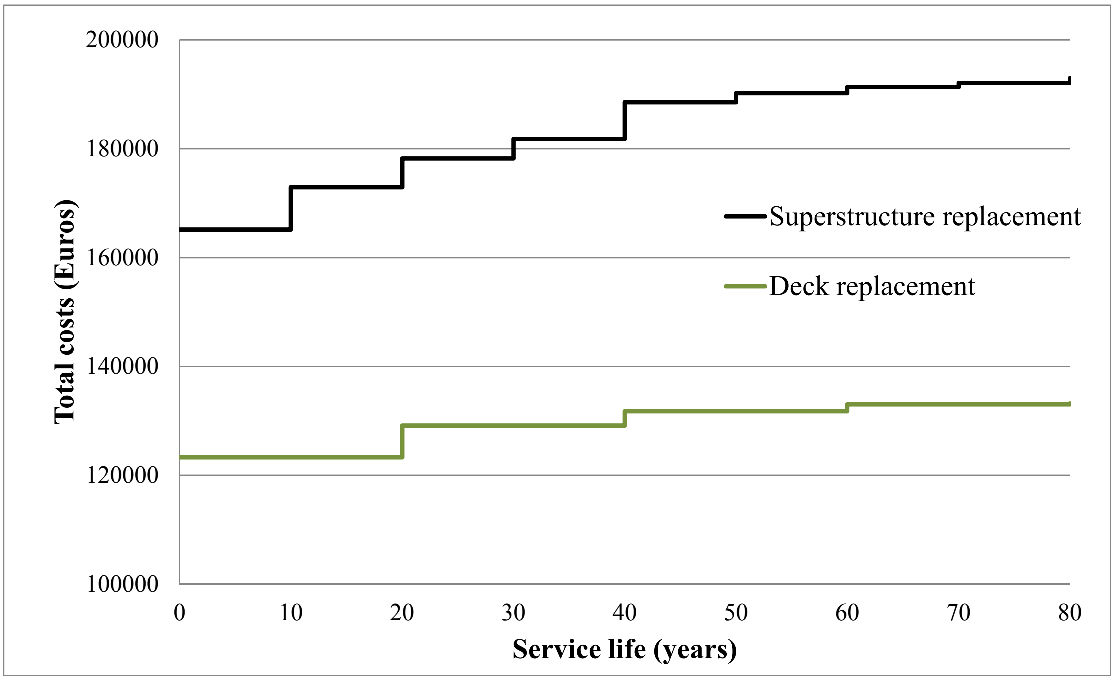


Figure 3.7 – Total costs over the assumed service life of 80 years for both bridge alternatives

From Figure 3.7, it can be seen that the option of deck replacement with an FRP deck results in less initial costs and is less costly during the entire design life, due to less maintenance activities. In both cases, initial construction costs dominate the total life-cycle costs, while the end of life/disposal costs are negligible, as depicted in Figure 3.8.

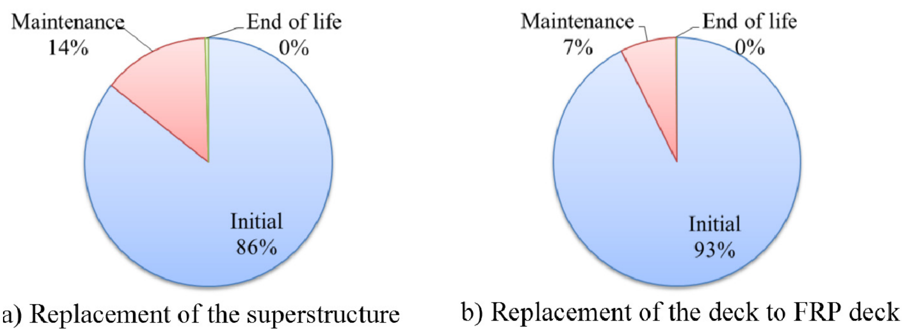
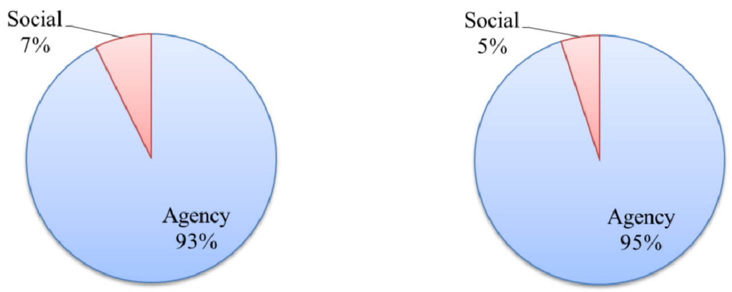


Figure 3.8 – Percentage of costs from different stages

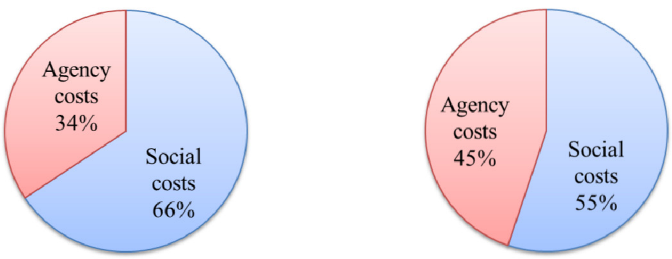
Comparing the agency costs to social costs, it is observed that for both cases the social costs comprise a small percentage of the total life-cycle costs (Figure 3.9). This is due to the low traffic volume passing the bridge.



a) Replacement of the superstructure b) Replacement of the deck to FRP deck

Figure 3.9 – Percentage of agency and social costs

A sensitivity analysis is performed for the parameter of average daily traffic volume by keeping all the other parameters constant, and it is observed that for a traffic volume of 20,000 vehicles per day the social costs comprise more than 50% of the total costs (refer to Figure 3.10).



a) Replacement of the superstructure b) Replacement of the deck to FRP deck

Figure 3.10 – Percentage of agency and social costs for ADT of 20,000

The life-cycle cost analyses show that the deck replacement option with an FRP deck is more favourable. In addition, the sensitivity analyses suggest that in more complex traffic situations with high traffic volumes, alternatives with a shorter construction time and fewer time-consuming maintenance activities are favoured.

3.3.3 Life-cycle assessment in terms of carbon emissions

Throughout the life cycle of a bridge the main sources of carbon emissions are shown in Figure 3.11.

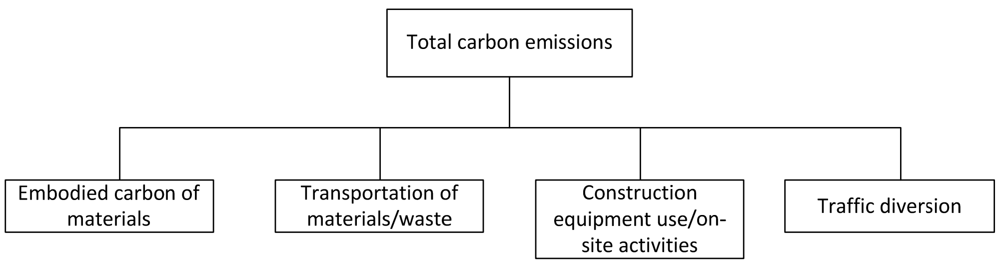


Figure 3.11 – Sources of carbon emissions and energy use throughout the life-cycle of a bridge

Embodied carbon of the materials is taken as the total primary energy used and carbon released including extraction of raw materials, manufacturing and transportation. The most common embodied carbon (as well as energy) of the materials follow a 'cradle-to-gate' boundary which includes all the energy consumed and carbon released until the product leaves the factory gate. It must be recognized that the values of embodied energy/carbon of materials can change significantly in different databases. One has to make sure that the same database sources are used when analyses are performed. The embodied energy and carbon of different materials commonly following 'cradle –to – gate' approach used for bridges are tabulated in Table 3.2.

Table 3.2 – Embodied energy and carbon of some materials used in bridge construction (Hammond and Jones 2008)

Materials	Embodied energy consumption (MJ/kg)	Embodied carbon emissions ($kgCO^2/kg$)
Concrete (general)	0.95	0.13
Steel (general)*	24.4	1.77
Steel (primary)	35.3	2.75
Steel (recycled)	9.5	0.43
Timber (general)	8.5	0.46
Glass fibres	28	1.53
GFRP	100	8.1

*The embodied energy and carbon for general steel is estimated by including recycled steel of 42.7%.

Such data is important to the designers to make careful material choices for sustainable design. However, it goes without saying that while evaluating the environmental impact of a material, the quantity and the functions of the materials need to be provided. For instance, even though the embodied energy consumption for concrete is less than GFRP, the quantity of GFRP in a bridge deck is substantially less than that in a concrete deck. In addition, it should be recognized that there is a certain uncertainty in these data, especially for GFRP. With its continuous development, the embodied energy and carbon of GFRP has reduced significantly. For example, Daniel (Daniel 2010) considers a value of 33MJ/kg for embodied energy consumption of pultruded GFRP members.

Transportation of the materials or waste causes carbon dioxide emissions. Transportation is included in several stages of the life-cycle of a bridge:

- Transportation of materials from the factory to the site during initial construction.
- Transportation of materials from the factory to the site during maintenance activities.
- Transportation of waste materials during maintenance/replacement and demolition.

The mode of transportation results in different amounts of carbon emissions (see Table 3.3). In addition to transportation of materials, the personnel travel from and to site can be considered as well.

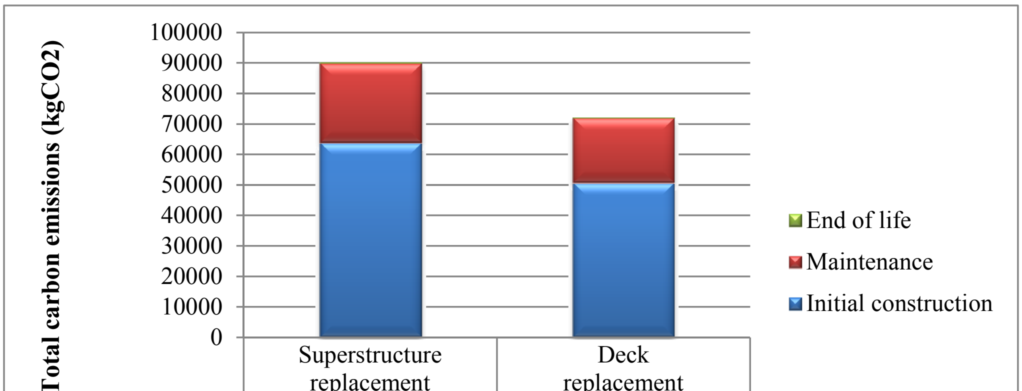
Table 3.3 – Carbon emissions for different modes of transportation ((EA) 2007)

Transportation	Unit amount of CO_2 emissions ($kgCO_2$/t km)
Road	0.1067
Water	0.015
Rail	0.037

On-site activates consume energy during construction, maintenance or demolition of a bridge, which in turn release carbon emissions. Depending on the types of activities and the construction equipment used, the energy consumption and carbon emissions can contribute considerably to the total carbon emissions.

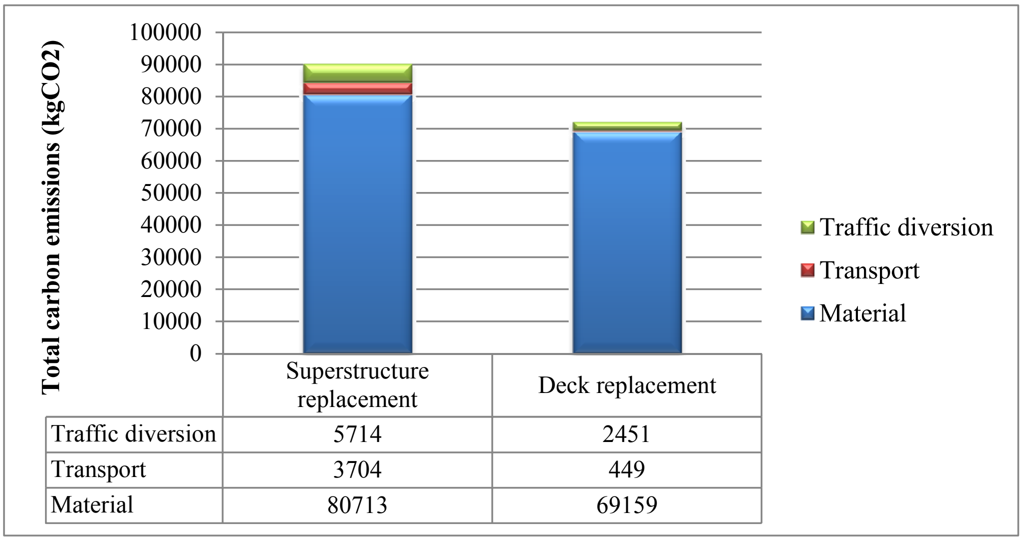
Another important source of carbon emissions is traffic diversion during construction, maintenance or disposal phases. Disruption of traffic, detouring and increased driving distances result in increased carbon emissions from fuel consumption of the cars. In urban areas, where average daily traffic is high, traffic disruptions might dominate the total carbon emissions.

In this study, the carbon emissions for the case-study bridge were calculated. Except the sources of carbon emissions from the construction equipment use/on-site activities, all the other sources were included. The results are presented in Figure 3.12 and Figure 3.13 according to the life-cycle stages and carbon emission sources respectively.



	Superstructure replacement	Deck replacement
End of life	246	49
Maintenance	26090	21292
Initial construction	63796	50718

Figure 3.12 – Total carbon emissions with respect to bridge life-cycle stages



	Superstructure replacement	Deck replacement
Traffic diversion	5714	2451
Transport	3704	449
Material	80713	69159

Figure 3.13 – Total carbon emissions with respect to carbon emission sources

The dominant carbon emissions occur during the initial construction, whereas the end-of-life carbon emissions are almost negligible. With respect to the carbon emission sources the embodied carbon emission of the materials covers more than 90% of the total carbon emissions. However, this result changes when the traffic volume increases. In Figure 3.14, the percentage of carbon emissions according to the sources for average daily traffic (ADT) 796 and 20,000 for both alternatives is compared. It is observed that when the average daily traffic increases the carbon emissions due to traffic detours increases.

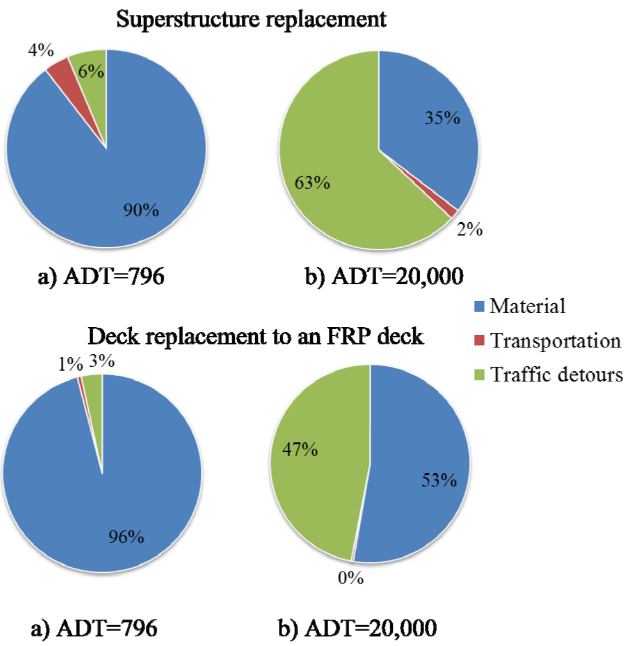


Figure 3.14 – Carbon emissions according to the sources for different average daily traffic (ADT)

In total, the results of the carbon emissions of the second alternative - replacement of the deck to an FRP deck - yield a lower value than the entire replacement of the superstructure to a new concrete-steel superstructure.

3.4 Summary and conclusions

In this case study, the replacement of the concrete deck with an FRP deck was the winning option with respect to life-cycle costs and carbon emissions compared with the replacement of the superstructure with a new prefabricated concrete-steel superstructure.

The traffic volumes play a significant role in both the life-cycle costs and the carbon emissions. If a large traffic volume is affected by traffic disruptions, the user cost and the carbon emissions will increase accordingly. Therefore, to offset the high initial cost and the embodied carbon emissions of FRP decks, it would be better if the target bridges are located on high-volume roadways with complex traffic situations.

Additional life-cycle analyses addressing environmental impact other than carbon emissions - such as: acidification potential (AP), eutrophication potential (EP), abiotic depletion potential (ADP), ozone depletion potential (ODP), ecotoxicity (ETC), human toxicity (HTC) etc. - should be performed on other bridge cases.

4 A novel joint for panel-level connections of FRP decks

4.1 Introduction

A detailed review of the connection types used for FRP decks in bridge construction is presented in Paper I. As a conclusion of this review the connection details for FRP composite decks require further development to ensure efficient and rapid on-site assembly.

In this thesis, focus is put on panel-level connections for the FRP decks. The conventional way of connecting FRP decks on-site is adhesively bonded connections. Even though these connections have performed well structurally up to now, they involve some shortcomings. The main shortcoming is the rather long time needed for the adhesive to cure and develop full strength (usually 48 hours at 20^{o}C). The required time for the adhesive to obtain its full strength would be translated to the time over which the bridge is out of service for traffic, which leads to prolonged traffic disturbances. To overcome this problem, a solution could be considering mechanical connections that do not use adhesives. Motivated by this, an innovative panel-level connection for FRP decks was developed, as shown in Figure 4.1.

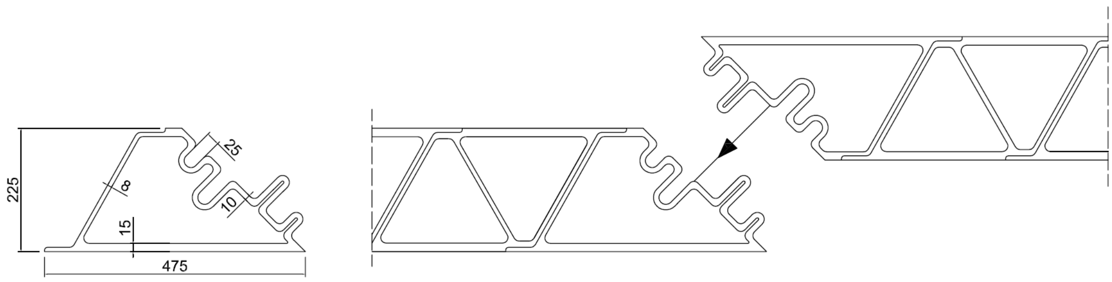


Figure 4.1 – Geometry and dimensions of the connection module (left) and configuration of the structural assembly (right)

The developed concept was the result of an interaction between the client, designer, contractor and manufacturer. The client in this thesis is referred to the bridge owner. The process followed for the development of the connection is shown in Figure 4.2 and Figure 4.3.

Firstly, the client requirements and their importance on a scale of 1-9 (1: weak, 9: strong) were defined. Afterwards, the constraints were defined mainly by the manufacturer. Based on the client requirements and the constraints, the design requirements were identified. In identifying the design requirements, the concept of 'design for manufacturing and assembly' (DFMA) was employed as well. DFMA concept is extensively used in industries such as automotive, aerospace etc., to provide guidance to the design team in simplifying the product structure, reducing manufacturing and assembly costs and quantifying improvements. The application of this method is relatively new and not well-established in construction industry, even though it has a great potential especially for the development of connections in bridges.

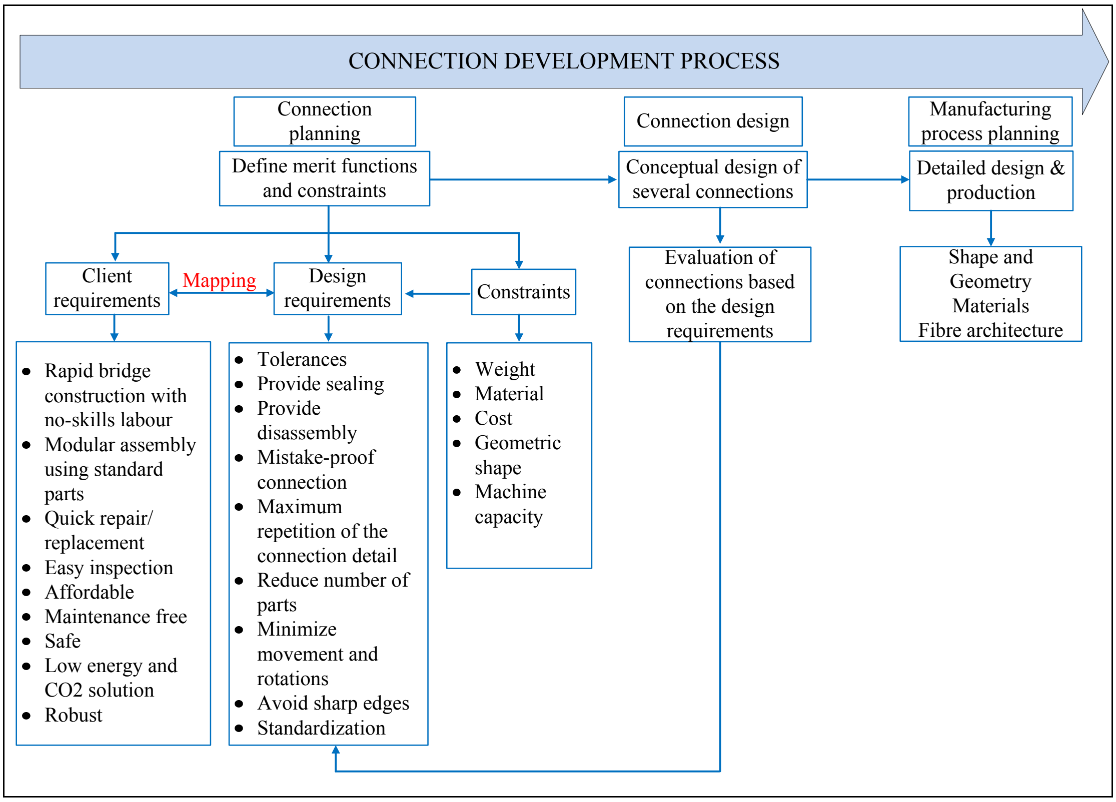


Figure 4.2 – The process taken for the development of connections

The design requirements were mapped with the client requirements in order to determine the priorities in the connection development process and to evaluate the connections concepts considered in the study (refer to Paper III). The mapping was performed by means of a matrix, where the relationship between the client and the design requirements was determined by answering the question: 'what is the strength of the relationship between the functional requirement and the design parameters? '. The relationship can be strong, medium or weak and carry a numeric value of 9, 4 or 1 respectively. Even though the scoring can be subjective it gives a general idea of where the priorities in the design must be given. The result of this matrix was the relative importance of each design requirement.

Several connection concepts were developed and evaluated by means of a second matrix as shown in Figure 4.3. The anticipated rate of fulfilling the design requirements of each developed connection was evaluated on a scale of 1 to 5. According to this evaluation, the winning connection concept was determined to be the one shown in Figure 4.1.

The last step was the detailed design of the winning connection concept for production, study it by means of numerical analyses and verify the design through experimental testing.

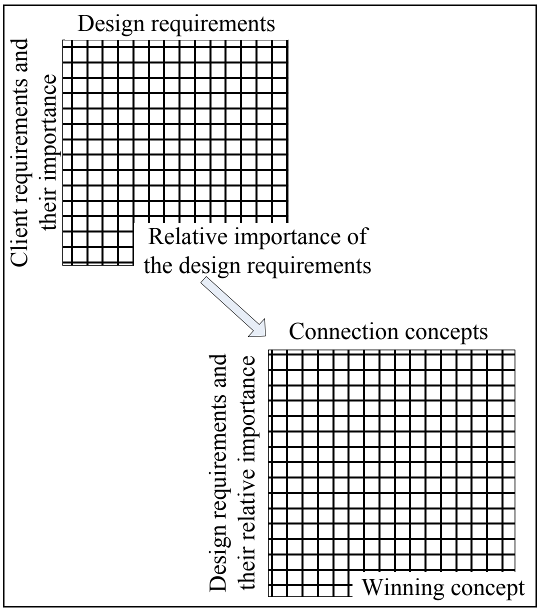


Figure 4.3 – Phases for the evaluation of the developed connection concepts

4.2 Design of the fibre architecture for the connection

The design of the fibre architecture (ply-stack up sequence) of the connection was necessary for the finite element modelling and the production. The criteria followed to design the fibre architecture were determined with the advice of the manufacturer as:

- Fibre orientations limited to 0^o, $\pm 45^o$ and 90^o
- Fibre type E-glass
- Matrix/ resin type epoxy

The selection of E-glass fibres was based on economic principles to provide a product which is economically feasible as well as to ensure compatibility with the FRP deck material.

Considering a fibre volume fraction of 52% after consulting with the manufacturer, the properties of a unidirectional (UD) E-glass-epoxy lamina were determined by mathematical models (rule of mixture, Halphin-Tsai method and the strength of material method (Agarwal et al. 2006)). It should be noted that the mathematical models are quite good in predicting the properties in the longitudinal direction of the lamina but they involve difficulty and are not very reliable for transverse properties of the composite lamina. Therefore, the computed properties by the mathematical models which are presented in Table 4.2 were slightly reduced.

The properties of the E-glass fibres and the epoxy resin considered for the design are presented in Table 4.1. The resulting properties of the UD lamina are tabulated in Table 4.2. The transverse compressive strength and the in-plane shear strength of the lamina were adopted from the literature (Agarwal et al. 2006) due to the limitations of the mathematical models. The UD fibre-reinforced lamina is usually assumed as transversely isotropic and the elastic constants are reduced to five as:

E_1; $E_2=E_3$; $\nu_{12}=\nu_{13}$; $G_{12}=G_{13}$; $G_{23}=E_2/2(1+\nu_{23})$

Table 4.1 – The properties of E-glass fibres and the resin

Property	**E-glass fibre**	**Epoxy resin**
Fibre volume fraction (%)	52	48
Density (g/cm^3)	2.54	1.2
Tensile modulus (GPa)	70	3
Tensile strength (MPa)	2600	70
Ultimate tensile strain	0.037	0.023*
Poisson ration	0.22	0.32
Shear modulus (GPa)	28	1.1

* the resin is assumed elastic

Table 4.2 - The mechanical properties for UD glass-epoxy lamina

Property	**Value**
E_x [MPa]*	37000
E_y, E_z [MPa]	10000
G_{xy}, G_{xz} [MPa]	4500
G_{yz} [MPa]	2900
ν_{xy}, ν_{xz}	0.26
ν_{yz}	0.72
f_{xu} [MPa]*	1200
f_{yu} [MPa]	30
f_{cxu} [MPa]*	-540
f_{cyu} [MPa]	-120
f_{sxy} [MPa]*	75

*X: fibre direction, Y: transverse direction
f_{xu}, f_{yu}: tensile strengths, f_{cxu}, f_{cyu}: compression strengths, f_{sxy}: ultimate in-plane shear strength

The resulting mechanical properties of the UD lamina were taken into account for the ply-stack up sequence design.

The design of the ply stack-up was based on finite element method. A finite element model was developed and the material of the connection module was assigned as isotopic. The direction and magnitude of the principal stresses for different load cases were determined. Thereafter, the direction of the fibres was determined to be aligned in the direction of the principle stresses in proportion to the magnitude of stresses. After determining the direction and proportion of the fibres according to the principal stresses, the design of the ply stack-up sequence was determined by taking into consideration several other criteria based on consultation with designers in aerospace engineering and the literature (Olsson 2006) as follows:

- Add plies in $\pm 45^o$ angle to the principle stresses to account for unexpected loads
- Include a minimum fraction of 90^o plies to avoid failure by unexpected transverse loads
- It is suggested to not use 0^o plies on unprotected surfaces to limit the effect of surface scratches.
- Use repeated sub laminates rather than thick plies which are more susceptible to cracking.
- Use symmetric (balanced) laminates to reduce buckling loads, warpage, twisting

from residual stresses.
- Avoid too large jumps in fibre orientation angles between different plies, in order to minimize the interlaminar shear stresses.
- To provide a robust structure, it is suggested to provide approximately 10% more fibres in each direction (mainly 0, ±45 and 90^{o}) than needed.

The final design of the ply stack-up sequence of the connection is presented in Figure 4.4. The flange and the web of the connection have a thickness of 15 mm and 10 mm respectively. The direction of the fibres denoted as 0^{o} is in the X-direction as shown in the figure. It should be noted that the coordinate system changes according to the shape of the connection. For example, in the circular areas of the connection the X-coordinate system (fibres aligned in 0^{o}) is the tangential axis of the circle. In Figure 4.4, the ply stack-up represents the direction of the fibres and the thickness of the laminas. The CSM lamina stands for continuous strand mat layer.

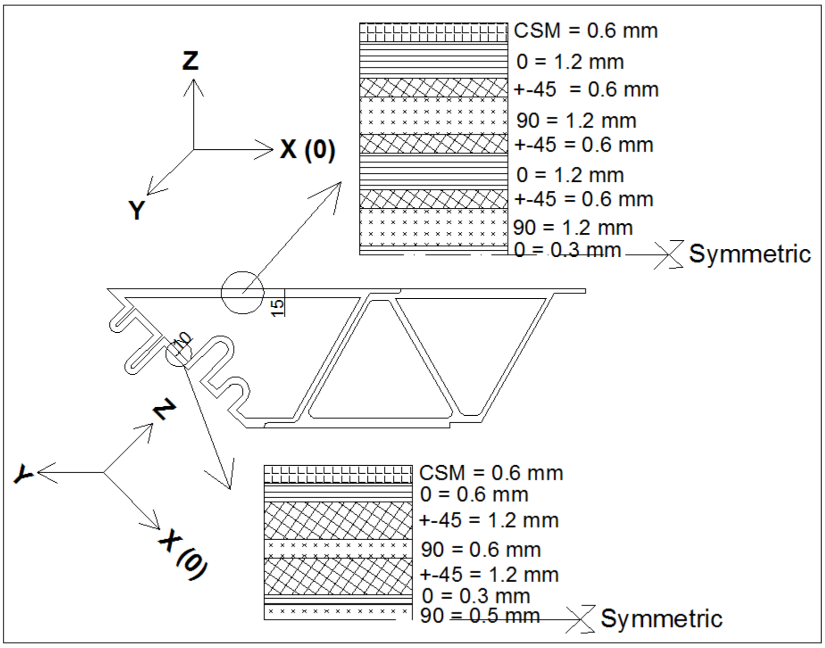


Figure 4.4 – Ply-stack up sequence of the connection module

The designed ply-stack up sequence of the connection was further used for the finite element analyses to investigate the overall performance of the developed connection described in the subsequent section.

4.3 Finite element analysis

The main aim of conducting the finite element analyses in this study was to investigate the overall behaviour of the developed connection and form the basis for the experimental testing. Detailed information regarding numerical modelling and results is presented in Paper III. It should be noted that the finite element model could not be validated from the experimental test results due to inconsistency of the delivered connection module from the manufacturer. The differences between the finite element model and the tested specimen are summarised in Table 4.3.

Table 4.3 – Differences between the FE-model and the tested specimen

Finite element model	*Tested specimen*
The contact between the connection modules was modelled as frictionless	In reality, friction between the connection modules exists
Perfect fit between the connection modules	Gaps were present between the connection modules
Material input data of the connection given in Table 4.2	Limited material data
Boundary conditions continuous and the uplift was locked	Boundary conditions discrete and the uplift of the specimen was free

4.4 Experimental investigation

The framework of the experimental study, including loading, instrumentation and results can be found in Paper III. Only one specimen was tested but in different load configurations. The tests were distinguished as serviceability load limit test in three different load positions and a final test (designated as T3-Fail) up to 433 kN load.

4.4.1 Serviceability load limit (SLS) tests

In the SLS tests the specimen was loaded up to 150 kN – which represents one wheel load - in three different positions as shown in Figure 4.5. Based on the load position, the SLS tests are designated as T1-SLS, T2-SLS and T3-SLS.

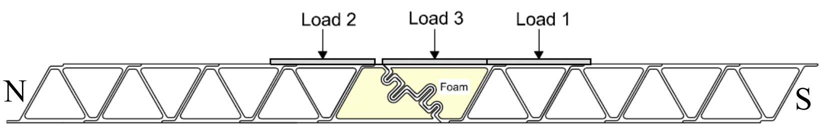


Figure 4.5 – Load positions for the testing

The load versus top flange deflection curves obtained for the three load positions in the experimental tests are shown in Figure 4.6 and are compared with the FE-results. As noted in Figure 4.6, the load-deflection curves show fairly good linear-elastic behaviour except T2-SLS test. The load-deflection curves from the FE-analyses are also linear. The non-linear behaviour in T2-SLS is attributed to the misfit of the produced connection modules as well as the engagement mechanism of the two panels.

In T1-SLS a perfect linear behaviour is observed since the South part of the connection dominates the North part and the connection modules cannot separate at any time, so the stiffness remains constant. On the other hand, when the load is applied on location 2, after a while, the North part of the connection cannot pull down the South part and therefore a separation takes pace. This separation has a gradual nature which results in loss of stiffness and displays a non-linear behaviour in T2-SLS.

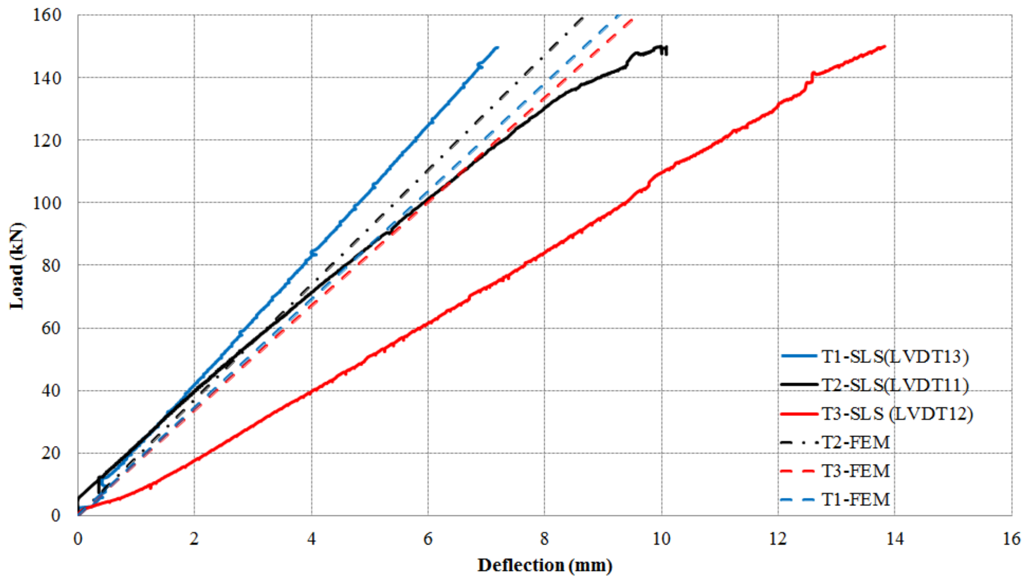


Figure 4.6 – Load-deflection curves for the SLS tests compared with results from FE-analyses

It is noted in Figure 4.6 that the stiffness of the specimen is dependent on the load position. This is attributed to the engagement of the specimen in the load-carrying capacity for each load position. Based on the reaction forces which are presented in Paper III, the approximate effective widths of the specimen carrying the applied load for each load position are shown in Figure 4.7. It is noticed that the stiffness of the specimen is related to the effective width of the specimen; higher effective width leads to higher stiffness.

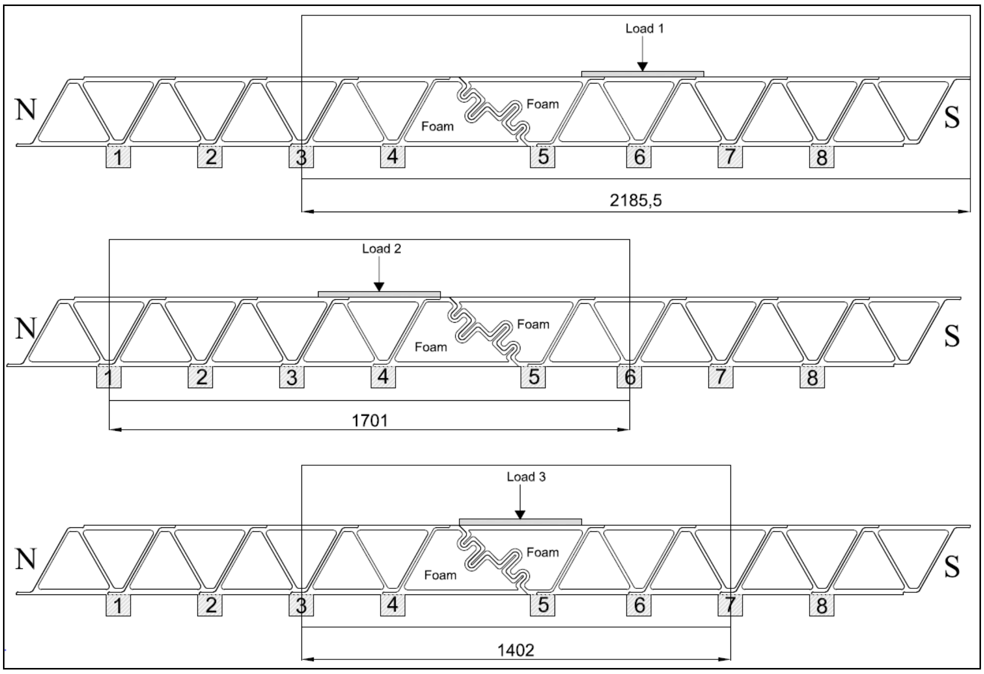


Figure 4.7 – Effective widths of the specimen for each load case

However, the FE results show that the stiffness of the specimen is comparable for each load case, which is due to the difference in the boundary conditions in the FE-analyses. Another observation is that in T1-SLS test the stiffness of the specimen is 17% higher than the FE-results. This is attributed to the difference of the material and geometrical properties of the FE-model and the connection module. As mentioned previously, the material properties of the manufactured connection module were lacking.

In Figure 4.6, a maximum deflection of 14 mm is recorded in T3-SLS test which exceeds the deflection limit of L/300 (10 mm) in Serviceability Limit State (SLS).

Another observation in the SLS tests is the vertical deflection incompatibility between the connection modules at the top flange, which might cause cracking of the wear surface. The vertical deflections in the longitudinal (span) direction of the specimen are presented in Figure 4.8 at the load levels of 50kN and 150kN for load position 2. There were no measured data for LVDT 7 and 21. It is noted in Figure 4.8 that at the load level of 50 kN, the magnitudes of the deflections at each side of the connection are close to each other, whereas at the load level of 150 kN differences in the vertical deflections on each side of the connection are observed due to separation of the deck panels. This difference in the top flange (around 4 mm) might pose a risk of wear surface cracking. On the other hand, it should be mentioned that the connection modules had gaps at the top flanges, which is discussed in Paper III.

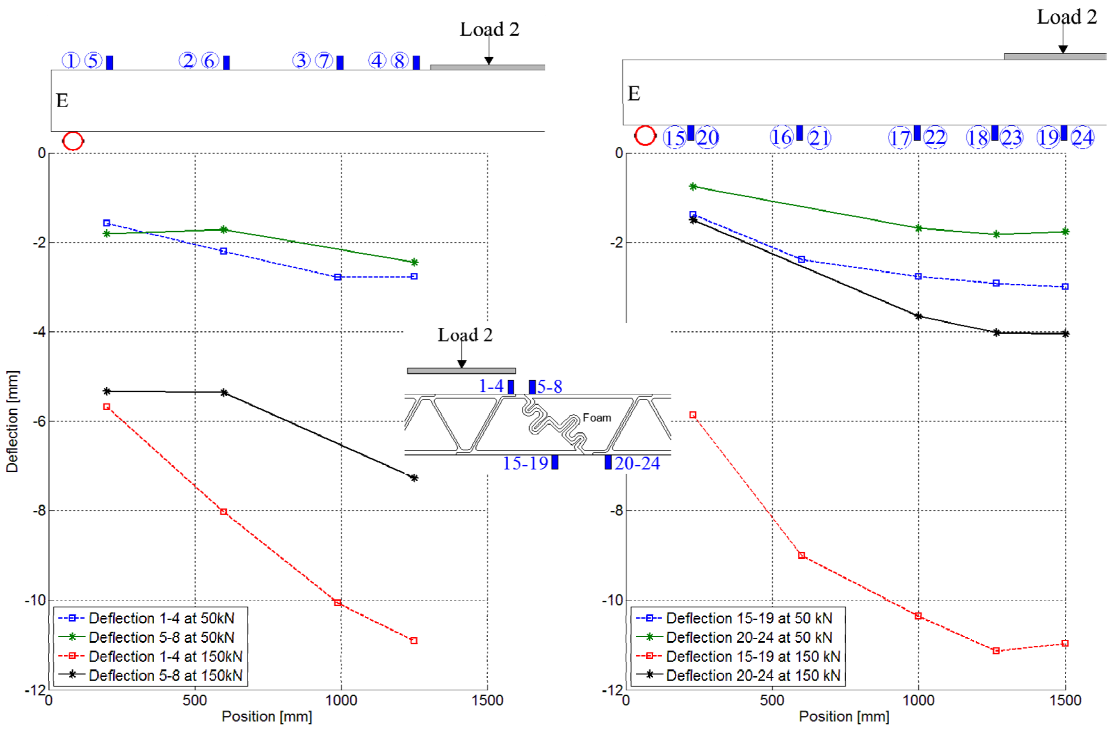


Figure 4.8 – Longitudinal deflection in the top (on the left) and bottom (on the right) flanges for load position 2

The specimen was instrumented also with strain gages and based on these strains the maximum stresses in the ASSET deck and the connection module flanges were calculated. The material properties of the ASSET deck and the connection flanges used for calculation of the stresses are tabulated below.

Table 4.4 – Material properties of the ASSET deck flanges and connection module

Property	E_L [MPa]	E_T [MPa]	ν_{LT}	ν_{LT}
ASSET deck	23000	18000	0.3	0.235
Connection module	22700	22700	0.3	0.3

The maximum obtained stresses for all load positions in the longitudinal and transverse stresses are summarised in Paper III.

Additional results regarding deflections, support reactions and the behaviour of the specimen are discussed in Paper III.

4.4.2 T3-Fail test

In the T3-Fail test, the specimen was loaded up to a maximum load of 433 kN in load position 3 (see Figure 4.5). The test was stopped at 433 kN due to load limits of the hydraulic jack. The deflection-load curves for different displacement transducers (LVDTs) in the top flange are shown in Figure 4.9. It is observed that the specimen can carry considerably more load than the ultimate limit state (ULS) load which is specified in Figure 4.9.

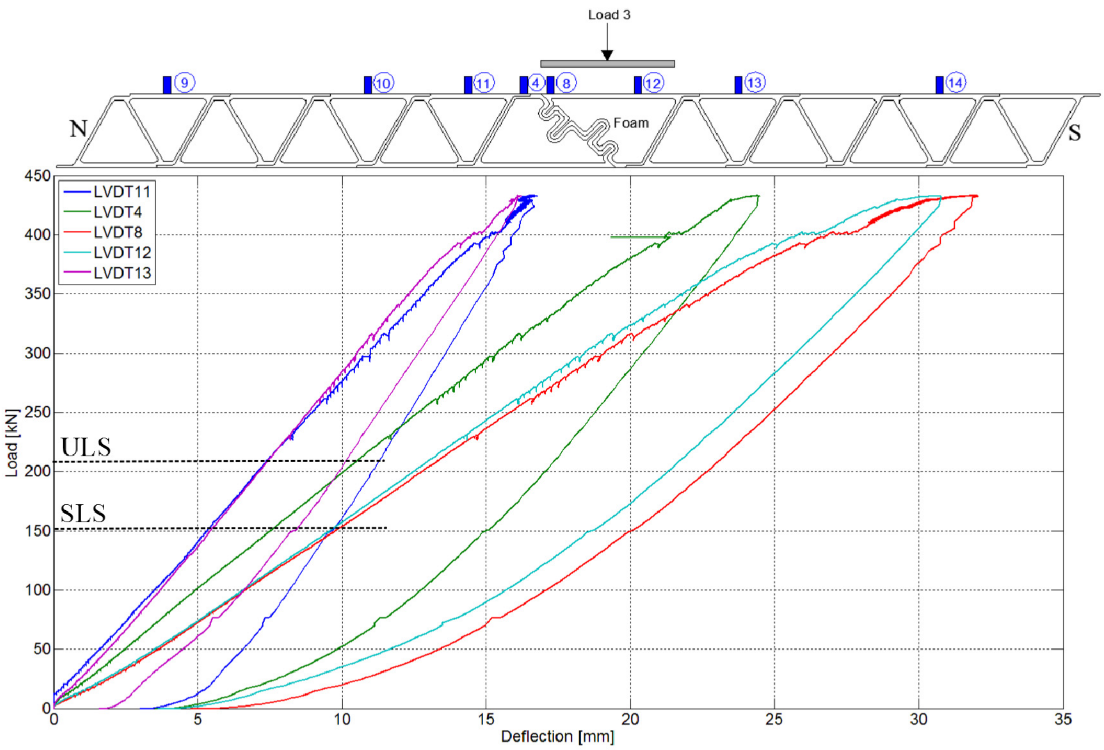


Figure 4.9 – Load-deflection curves for the top flange for the final test

The load-deflection curves display a fairly linear behaviour up to a load of approximately 250 kN and then a slightly non-linear pattern is observed. After a load of 400 kN this non-linear behaviour is more pronounced. At a load of 315 kN the first delamination failures in the flanges of the ASSET deck were observed, shown in Figure 4.10. More information about the delamination failures is described in Paper III.

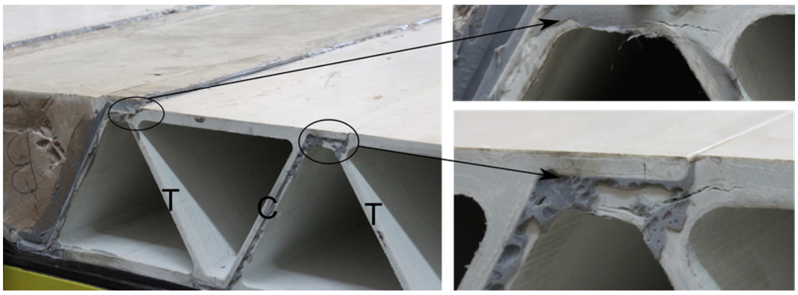


Figure 4.10 – Delamination of the lap joint of the ASSET deck at a load of 315 kN

The measured strains in the longitudinal direction of the specimen in the top and bottom flanges for different load levels are illustrated in Figure 4.11. The strains in the ASSET deck in the south part (9-12, 20-23) were higher than in the north part. This indicates that the south part of the specimen carried more loads, which was actually the part where most of the delamination failures occurred.

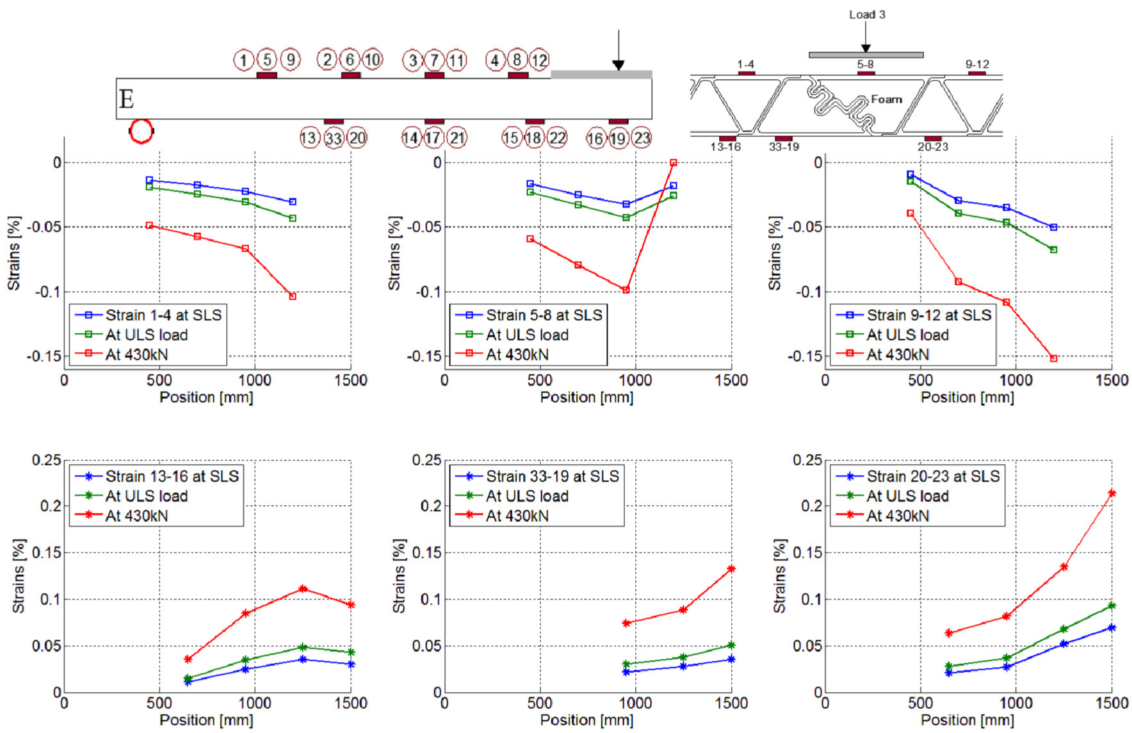


Figure 4.11 – Top and bottom strains in the longitudinal direction of the specimen at different load levels

Based on the measured strains, the maximum longitudinal and transverse stresses in the ASSET deck flanges were computed at different load levels and are summarised in Table 4.5. It is observed that the stresses are around 6 times lower than the strength of the material. However, as the failure mode shows, it is not these stresses which are critical for the ASSET deck. The failure is governed by through-thickness tensile stresses of the flanges.

Table 4.5 – Computed stresses in the flanges of the ASSET deck and the strength properties at different load level for the final test (all units in MPa)

	Top flange		Bottom flange		Strength			
	σ_L	σ_T	σ_L	σ_T	f_{Lu} *	f_{Tu}	f_{cLu} *	f_{cLu}
At 202 kN	-15.8	-0.67	22.3	3.07	300	220	-250	-200
At 350 kN	-27.2	-1.4	39.2	8.1				
At 430 kN	-36.3	-4.8	53.8	15.3				

*L: pultrusion direction, T: transverse direction,
f_{Lu}, f_{Tu}: tensile strengths, f_{cLu}, f_{cTu}: compression strengths

4.5 Summary and conclusions

In this study, a novel connection is proposed for fibre-reinforced polymer bridge deck panels with potential for fast on-site assembly. Finite element modelling and static experimental tests were performed to examine and verify the structural behaviour. Based on the analyses and the results presented also in Paper III the following conclusions can be drawn:

- In the SLS experiments, the load-deflection responses were mainly linear-elastic. In one of the tests, a slight non-linear behaviour was observed which is attributed to the misfit between the connection modules as well as the engagement mechanism of the connection modules. Due to the misfit and gaps between the connection modules, an abrupt difference in deflection was observed at the top flanges of the connection modules which might be a threat for wear surface cracking. Therefore, specimens with tight tolerances with an applied wear surface are recommended for testing.
- In the final test, a pseudo-ductile behaviour was observed in the load-displacement curves. Progressive delamination failures started in the ASSET deck flanges at a load of 315 kN due to the tensile forces carried by the webs. These tensile forces cause through-thickness tensile stresses in the intersections of the webs and the flanges, resulting in delamination between the fibre lay-ups in the flanges. At this load level, the stresses in longitudinal and transverse direction of the ASSET deck flanges remained quite low compared with the ultimate strength. This shows that the FRP laminates are sensitive to through-thickness tensile stresses and control the strength of the material. However the delaminations started at a load around 50% higher than the ultimate limit state load.
- Failure of the specimen was not reached at 433 kN meaning that the failure load of the specimen is higher than 433 kN.
- Further tests are required to examine the fatigue performance and the environmental effects on the durability of the connection.

5 Conclusions

The background related to fibre reinforced polymer (FRP) bridge decks was presented to provide the state-of-knowledge and identify possible knowledge gaps within the field of FRP decks. This background provided the framework and the motivation of this thesis, which involved two research areas: i) to analyse the sustainable potential of bridges with FRP decks by conducting life-cycle cost analyses and life-cycle assessment and, ii) to develop a novel panel-level connection for FRP decks with focus on swift on-site assembly and investigate its performance.

A detailed assessment of the life-cycle cost and the environmental impact in terms of carbon emissions related to a case-study bridge was conducted. The total replacement of a steel-concrete bridge with a bridge rehabilitation scenario in which the concrete deck was replaced by an FRP deck was compared. The results indicated that the scenario of the replacement of the deteriorated concrete deck with an FRP deck yields less life-cycle costs and carbon emissions than the replacement of the bridge with a new steel-concrete bridge. In addition, the increase in average daily traffic (ADT) passing the bridge significantly influences both the cost and the carbon emission results, which in turn is favourable for the FRP deck alternative thanks to its fast installation. Additional life-cycle cost analyses and life-cycle assessment are suggested for bridges with FRP decks in different conditions. For instance, life-cycle assessments including environmental impacts other than carbon emissions are necessary. Due to sparse and limited input data for both life-cycle cost and life-cycle assessments, the development of internationally-accepted databases is necessary.

A novel panel-level connection for FRP decks was designed conceptually and the structural behaviour was investigated numerically and experimentally. The connection was designed through collaboration between the designer, client, manufacturer and contractor. Focus was given to the development of a connection concept providing swift on-site assembly. After manufacturing, the connection was proven to be effective in regards to rapid installation. However, the method of manufacturing the connection modules in this study, namely vacuum infusion method, was not efficient and geometric flaws resulted in the connection modules. The presence of flaws jeopardized the performance of the connection modules in the serviceability limit state. Deflection inconsistencies were observed in the performed experimental tests at the top flanges of the connection modules which could pose a risk of wear surface cracking. The formulation of deflection limits based on the prevention of wear surface cracking is unavailable. Nevertheless, these deflection inconsistencies at the top flange were not observed in the finite element analyses which were carried out with perfect fit between the connection modules. Therefore, additional tests with specimens with accurate fit and an applied wear surface are necessary to reach conclusive statements for the serviceability limit state performance of the connection module. The load-carrying capacity of the tested specimen was higher than 433 kN which is more than twice the ultimate limit state load (202.5 kN). A slightly ductile behaviour was observed from the load-deflection curves. The results of this study show that the proposed connection has a good potential to be used for FRP decks. Additional tests regarding the fatigue performance and the environmental effects on the durability of the connections are suggested.

6 References

(AASHTO), American Association of State Highway and Transportation Officials. (2008). "Guide specifications for design of FRP pedestrian bridges."

(EA), Environment Agency. (2007). "Carbon calculator for construction activities." *Environment Agency, London*.

Agarwal, Bhagwan D., Lawrence J. Broutman, and K. Chandrashekhara. (2006). *Analysis and Performance of Fiber Composites*: John Wiley & Sons, 2006.

Agency, The Highway, Scottish Executive, Wellsh Assmebly Government, and The Department for Regional Development Northern Ireland. (2005). "Design of FRP Bridges and Highway Structures."

Alampalli, Sreenivas, and Jonathan Kunin. (2002). "Rehabilitation and field testing of an FRP bridge deck on a truss bridge." *Composite Structures* 57 (1-4):373-5.

Alampalli, Sreenivas, Jerome O'Connor, and Arthur P. Yannotti. (2002). "Fiber reinforced polymer composites for the superstructure of a short-span rural bridge." *Composite Structures* 58 (1):21-7.

Baumann, Henrikke, and Anne-Marie Tillman. (2004). *The Hitch Hiker's Guide to LCA*. Lund, Sweden: Studentlitteratur.

Canning, L., J. Hodgson, R. Karuna, et al. (2007). "Progress of advanced composites for civil infrastructure." *Proceedings of the Institution of Civil Engineers: Structures and Buildings* 160 (6):307-15.

Canning, Lee. (2012). Performance and 8-year load test on West Mill FRP Bridge. Paper presented at the CICE 2012, Rome, Italy.

Coogler, K., K. A. Harries, B. Wan, et al. (2005). "Critical evaluation of strain measurements in glass fiber-reinforced polymer bridge decks." *Journal of Bridge Engineering* 10 (6):704-12.

Daniel, R. A. (2010). "A composite bridge is favoured by quantifying ecological impact." *Structural Engineering International: Journal of the International Association for Bridge and Structural Engineering (IABSE)* 20 (4):385-91.

Domone, P., and J. Illston. (2010). *Construction Materials: Their Nature and Behaviour, Fourth Edition*: Taylor & Francis.

Du, Guangli. (2012). "Towards Sustainable Construction: Life Cycle Assessment of Railway Bridges." Licentiate Thesis, KTH Royal Institute of Technology.

Fu, C. C., H. Alayed, A. M. Amde, and J. Robert. (2007). "Field performance of the fiber-reinforced polymer deck of a truss bridge." *Journal of Performance of Constructed Facilities* 21 (1):53-60.

Gan, Lian Hong, Lin Ye, and Yu-Wing Mai. (1999). "Design and evaluation of various section profiles for pultruded deck panels." *Composite Structures* 47 (1-4):719-25.

Gervásio, H., and L. S. da Silva. (2008). "Comparative life-cycle analysis of steel-concrete composite bridges." *Structure and Infrastructure Engineering* vol. 4 (issue 4):p. 251-69.

Gervásio, H., and L. Simões da Silva. (2013). A probabilistic approach for life-cycle environmental analysis of motorway bridges.

GreenDelta GmbH. (2013). "openLCA." In.

Grimm, Evan. (2006). "Load distribution in FRP decking on timber stringer bridge system." Brigham Young University.

Haghani, Reza. (2011). "D5.3 - Needs for maintenance and refurbishment of bridges in urban environments." *PANTURA*.

Hammervold, Johanne, Marte Reenaas, and Helge Brattebø. (2009). "ETSI Stage 2 SUB-PROJECT 2: Environmental Effects - Life Cycle Assessment of Bridges." In.: Norwegian University of Science and Technology (NTNU).

Hammond, Geoff, and Craig Jones. (2008). "Inventory of Carbon and Energy (ICE) - Version 1.6a." *University of Bath*.

ISO. (2006). "Environmental management - Life cycle assessment - Principles and framework (ISO 14040:2006)." In.: International Organization for Standards.

Kalny, Ondrej, Robert J. Peterman, and Guillermo Ramirez. (2004). "Performance Evaluation of Repair Technique for Damaged Fiber-Reinforced Polymer Honeycomb Bridge Deck Panels." *Journal of Bridge Engineering*.

Karbhari, Vistasp M. (2001). "HITEC EVALUATION PLAN FOR FRP COMPOSITE BRIDGE DECKS."

Keller, T., and H. Gürtler. (2005). "Composite action and adhesive bond between fiber-reinforced polymer bridge decks and main girders." *Journal of Composites for Construction* 9 (4):360-8.

Keller, Thomas, and Herbert Gürtler. (2005). "Quasi-static and fatigue performance of a cellular FRP bridge deck adhesively bonded to steel girders." *Composite Structures* 70 (4):484-96.

Keller, Thomas, and Herbert Gürtler. (2006). "In-plane compression and shear performance of FRP bridge decks acting as top chord of bridge girders." *Composite Structures* 72 (2):151-62.

Keller, Thomas, and Martin Schollmayer. (2004). "Plate bending behavior of a pultruded GFRP bridge deck system." *Composite Structures* 64 (3-4):285-95.

Keller, Thomas, and Till Vallée. (2005). "Adhesively bonded lap joints from pultruded GFRP profiles. Part II: joint strength prediction." *Composites Part B: Engineering* 36 (4):341-50.

Kim, Hyeong-Yeol, Yoon-Koog Hwang, Ki-Tae Park, et al. (2005). "Fiber reinforced plastic deck profile for I-girder bridges." *Composite Structures* 67 (4):411-6.

Knippers, J., E. Pelke, M. Gabler, and D. Berger. (2010). "Bridges with glass fibre-reinforced polymer decks: The road bridge in Friedberg, Germany." *Structural Engineering International: Journal of the International Association for Bridge and Structural Engineering (IABSE)* 20 (4):400-4.

Kong, B., C. S. Cai, and X. Kong. (2013). "Thermal behaviors of concrete and steel bridges after slab replacements with GFRP honeycomb sandwich panels." *Engineering Structures* 56:2041-51.

Lee, S. W., K. J. Hong, and S. Park. (2010). "Current and future applications of glass-fibre-reinforced polymer decks in Korea." *Structural Engineering International: Journal of the International Association for Bridge and Structural Engineering (IABSE)* 20 (4):405-8.

Lewis, A. E., C. Dumlao, M. J. Lee, and T. L. Weinmann. (2004). "Performance of Schuyler Heim composite Bridge deck replacement port of Los Angeles, California." In, 36-46.

Lippiatt, B. C. (2009). Evaluating sustainability using standard approaches: The BEES tool.

Mara, Valbona, Mohammad Al-Emrani, and Robert Kliger. (2012). Upgrading of an existing concrete-steel bridge using fibre reinforced polymer deck- A feasibility study. Paper presented at the FRP Bridges Conference, London, UK.

Olsson, Robin. (2006). "Composite mechanics & laminate theory." In. Imperial College: Department of Aeronautics.

Reising, R. M. W., B. M. Shahrooz, V. J. Hunt, et al. (2004). "Close look at construction issues and performance of four fiber-reinforced polymer composite bridge decks." *Journal of Composites for Construction* 8 (1):33-42.

Sams, M. (2005). Broadway bridge case study bridge deck application of fiber-reinforced polymer.

Turner, M. K., K. A. Harries, M. F. Petrou, and D. Rizos. (2004). "In situ structural evaluation of a GFRP bridge deck system." *Composite Structures* 65 (2):157-65.

Zhou, A., J. T. Coleman, A. B. Temeles, et al. (2005). "Laboratory and field performance of cellular fiber-reinforced polymer composite bridge deck systems." *Journal of Composites for Construction* 9 (5):458-67.

Zhou, Aixi, John J. Lesko, Jason T. Coleman, and Thomas E. Cousins. (2002). "Failure modes and failure mechanisms of fiber reinforced polymer composite bridge decks."

Zhou, E., Y. Wang, D. Meggers, and J. Plunkett. (2007). "Field tests to determine static and dynamic response to traffic loads of fiber-reinforced polyester no-name creek bridge." In, 231-7.

Zimoch, Ewa, and Joan Ramón Casas Rius. (2012). "Life-cycle assessment (LCA) of existing bridges and other structures." Master's thesis, Escola Tècninca Superior d'Enginyers de Camins, Canals i Ports de Barcelona, Universitat politècnica de Catalunya.

Zureick, A. H., B. Shih, and E. Munley. (1995). "Fiber-reinforced polymeric bridge decks." *Structural Engineering Review* 7 (3):257-66.