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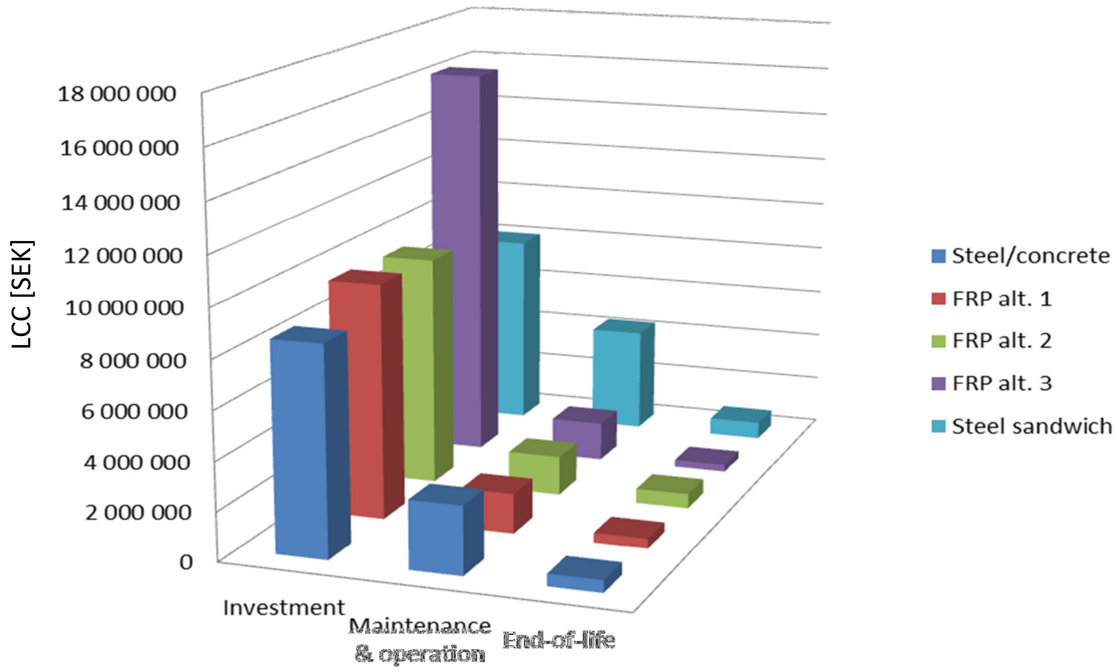

Comparative study of bridge concepts based on life-cycle cost analysis and life-cycle assessment

Master of Science Thesis in the Master's Programme Structural Engineering and Building Technology

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Department of Civil and Environmental Engineering
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CHALMERS UNIVERSITY OF TECHNOLOGY
Göteborg, Sweden 2013
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Cover:
Illustration of the distribution of costs over the life-cycle for different bridge concepts.

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ABSTRACT

Sustainable development has gained increasing focus in the bridge industry during recent years, regarding both economic and environmental impacts. In line with this, holistic approaches are needed which consider all costs and environmental impacts in a life-cycle perspective. This thesis aims to evaluate new materials and concepts in the bridge industry, from a sustainability point of view, by implementing life-cycle cost analysis and life-cycle assessment in a case study.

In the case study the competitiveness of three different bridge designs in terms of costs and environmental impact is evaluated. A design with a conventional steel/concrete composite bridge is compared to two relatively new concepts in which the concrete deck is replaced by a steel sandwich deck or a fibre reinforced polymer deck. For the latter bridge concept, three different design alternatives are considered. In the life-cycle cost analyses the net present value method is used. In the life-cycle assessment, ReCiPe and USEtox are used through the software tools openLCA and BridgeLCA. Furthermore, two different scenarios are evaluated for each bridge concept. Scenario 1 considers an entirely new construction, while scenario 2 evaluates the case when an existing bridge is to be replaced. Thus, the main difference between the scenarios is the amount of traffic disrupted during construction.

The results of the analyses show that the majority of the costs occur in the investment phase while the costs in the end-of-life phase are negligible. A sensitivity analysis for the life-cycle cost analyses shows that the total life-cycle cost is sensitive to changes of the discount rate, the traffic volume and the price of the fibre reinforced polymer deck. The conventional solution with a steel/concrete composite bridge has the lowest life-cycle cost as well as the least environmental impact in the first scenario, where little traffic is affected in the construction phase.

In the second scenario no obvious winner is found based solely on the life-cycle cost and environmental impact. In the results of the life-cycle assessment a concept with fibre reinforce polymer deck has the lowest emissions in four out of five categories. However the steel/concrete composite bridge has a lower eutrophication impact which dominates the final result. With this in mind, one of the designs alternatives with a fibre reinforced polymer deck is recommended in scenario 2 since it is deemed to have potential for development, especially in a complex traffic environment.

Key words: life-cycle cost, LCC, life-cycle assessment, LCA, bridge, deck, fibre reinforced polymer, steel sandwich

Jämförande studie av brokoncept baserat på livscykelkostnadsanalys och livscykelanalys

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SAMMANFATTNING

Fokus på hållbar utveckling har ökat inom brobyggandet under de senaste åren, både med hänsyn till ekonomiska och miljömässiga aspekter. Som ett led i detta behövs metoder med ett helhetstänkande där både kostnader och miljöpåverkan under livslängden beaktas. Målet med studien är att utvärdera nya material och koncept för brobyggande, ur ett hållbarhetsperspektiv, genom att utföra livscykelkostnadsanalyser och livscykelanalyser i en fallstudie.

I fallstudien utvärderas konkurrenskraften för tre olika brotyper i form av kostnad och miljöpåverkan. En konventionell samverkansbro i stål och betong jämförs med två relativt nya koncept där betongdäcket ersätts med ett stålsandwichdäck eller ett däck av fiberarmerade polymerer. För den senare brotypen analyseras tre alternativa koncept. Livscykelkostnadsanalysen tar hänsyn till både agenturkostnader och användarkostnader och utförs med hjälp av nuvärdesmetoden. I livcykelanalysen används metoderna ReCiPe och USEtox genom programvarorna openLCA och BridgeLCA. Vidare undersöks två olika scenarier för varje brokoncept. Scenario 1 analyserar en helt ny konstruktion medan scenario 2 utvärderar fallet då en existerande bro ska bytas ut. Den huvudsakliga skillnaden är alltså mängden trafik som utsätts för störningar under uppförandet.

Resultaten från analyserna visar att majoriteten av kostnaderna uppstår under investeringsfasen medan kostnaderna i rivningsfasen blir försumbara. En känslighetsanalys av livscykelkostnadsanalysen visar att den totala livscykelkostnaden i stor utsträckning påverkas av diskonteringsräntan, trafikvolymen och priset på det fiberarmerade polymerdäcket. Den traditionella lösningen med en samverkansbro har den lägsta livscykelkostnaden samt den minsta miljöpåverkan i det första scenariot, där små trafikmängder påverkas under uppförandefasen.

I det andra scenariot fanns ingen tydlig vinnare, baserat enbart på kostnader och miljöpåverkan. Resultaten från livscykelanalysen visar att ett av koncepten med däck av fiberarmerade polymerer ger minst miljöpåverkan i fyra av fem kategorier. Dock har samverkansbron lägre påverkan på övergödning, vilket dominerar resultatet. Med detta i åtanke, rekommenderas ett av koncepten med däck av fiberarmerade polymerer i scenario 2 då det bedöms ha en stor utvecklingspotential, i synnerhet i komplexa trafikmiljöer.

Nyckelord: livscykelkostnad, LCC, livscykelanalys, LCA, bro, brodäck, fiberarmerade polymerer, stålsandwich

Contents

Preface

This thesis was written at the Department of Civil and Environmental Engineering, division of Structural Engineering, at Chalmers University of Technology during the spring 2013. The thesis was conducted in cooperation with COWI and Trafikverket and is part of the European research project PANTURA, for sustainable urban constructions.

We would like to thank our supervisors Valbona Mara at Chalmers, Dan Nilsson, Magnus Bäckström, and Niklas Larsson at COWI, Karin Mehlberg at Trafikverket and our examiner Mohammad Al-Emrani at Chalmers for all the help and support we have received during our work.

During our work with this thesis several questions have arisen, requiring the knowledge of professionals. Therefore we would like to thank:

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Göteborg, June 2013

Amanda Sagemo
Linnea Storck

List of abbreviations

ADT	Average Daily Traffic
AP	Terrestrial acidification potential
ASTM	American Society for Technology and Materials
BaTMan	Bridge and Tunnel Management system
BEES	Building for Environmental and Economic Sustainability
EP	Freshwater eutrophication potential
ET	Ecotoxicity
ETSI	Bridge life-cycle optimisation (*Elinkaareltaan tarkoituksenmukainen silta)*
FD	Fossil depletion
FRP	Fibre reinforced polymer
GMAW	Gas Metal Arc Welding
GWP	Global warming potential
HTC	human toxicity cancer
HTNC	human toxicity non-cancer
ISO	International Organization of Standardization
LCA	Life-cycle assessment
LCC	Life-cycle cost
LCCA	Life-cycle cost analysis
LCI	Life-cycle inventory
LCIA	Life-cycle impact assessment
MADA	Multi-attribute decision analysis
MR&R	Maintenance, rehabilitation and repair
NIST	National institute of standards and technology
NPV	Net present value
OP	Ozone depletion potential
PMMA	Polymer modified mastic asphalt
SETAC	Society of Environmental Toxicology and Chemistry
SVID	Swedish Industrial Design Foundation (Stiftelsen svensk industridesign)

Notations

Upper case letters

A_a	bridge accident rate during the work activities
A_n	bridge accident rate during normal conditions
ADT_t	average daily traffic at time t
C_{ACC}	accident cost
$C_{acquisition}$	costs for purchasing, construction and installation
C_{agency}	expenses of the owner
C_F	average cost per fatality for the society
C_I	average cost per serious injury accident for the society
$C_{MR\&R}$	costs for inspections, operation, maintenance, repair and disposal
C_n	sum of all cash flows in year n
C_{TDC}	travel delay cost
C_{VOC}	vehicle operation cost
Δt_{mean}	mean traffic delay time
E_{xx}	Modulus of elasticity in x-direction
E_{yy}	Modulus of elasticity in y-direction
I_{xx}	Moment of inertia around the x-axis
I_{yy}	Moment of inertia around the y-axis
L	service life-span
N_t	number of days of road work at time t
NPV	life-cycle cost expressed as a present value
O_p	average hourly operating cost for one passenger car
O_T	average hourly operating cost for one truck including its goods operation
P_F	average number of persons killed in bridge-related accidents
P_I	average number of persons injured (not killed) in bridge-related accidents
T	travel time delay for one vehicle

Lower case letters

b_f	flange width
c_{jk}	characterization indicator for stressor *j* to impact category *k*

d_k	total potential impacts in impact category *k*, expressed in equivalents
e_{ij}	emission of the stressor *j* for the total consumption of input parameter *i*
f_{ij}	emissions of stressor *j* per unit of input parameter *i*
h	height of beam
m_k	normalized potential impacts for category *k*
n	year when the cost occurs
n_k	normalization factor for category *k*
r	discount rate
r_T	percentage of trucks of the total ADT
s_{wz}	length of work zone
t_f	flange thickness
t_w	web thickness
v	weighted result, sum of all impact categories
v_{red}	assumed reduced speed
w_k	weighting factor for impact category *k*
w_p	hourly time value for one passenger car
w_T	hourly time value for one truck
x_i	consumption of input parameter *i*

1 Introduction

Sustainable development is 'the development that meets the needs of the present without compromising the ability of future generation to meet their own needs' (United Nations General Assembly, 1987). The concept of sustainable development considers economic, environmental and social sustainability and states the importance of a holistic perspective (Gervásio and da Silva, 2012). Actions should be in balance with nature, both in the present and in the future. Today, resource consumption is extensive all over the world and there is an increased awareness of the need to change the consumption patterns in order not to compromise with the needs of future generations. The construction sector is the single 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 economy and environment in balance.

Unnecessary costs and environmental impacts, due to short sighted thinking when making investment decisions, are contributing factors to the high resource consumption. This thesis is focused on bridge construction, where investment costs often are the only costs considered in the decision-making process and little attention is paid to environmental effects (Gu et al., 2009). Reducing costs is of special interest in the bridge sector since bridges often are funded by taxes and therefore their costs affect the entire society.

One way to consider sustainability is by using life-cycle thinking, which includes environmental, economic and social aspects over the whole life span of a product or service (Gervásio and da Silva, 2012, Du, 2012). The economic aspect can be considered by using life-cycle cost analysis (LCCA), which sums up the total life-cycle cost (LCC), including all costs from acquisition to demolition (Sterner, 2000). During procurement of bridges, LCCA facilitates choosing the alternative with the lowest total cost, regardless of the investment cost. Likewise, life-cycle assessment (LCA) is a tool which can be used to gather all environmental impacts of a product from cradle to grave. For infrastructure projects, it is especially important to consider costs and environmental impact for the whole life-cycle since they have a longer life span than most products (Gu et al., 2009). Access, amenity, user comfort and satisfaction, community health and welfare are among the parameters involved in the social aspects. Examples of social aspects are queues emerging during construction work or loss of income for shops affected by the construction (Davis Langdon Management Consulting, 2007a).

The concepts of LCC and LCA were developed during the 1960's and 1970's (Russell et al., 2005) but they have only recently been introduced in the construction sector and are not commonly used in this field (Safi et al., 2012). To change this, and consequently reduce the resource consumption, research is needed to point out the benefits of using life-cycle thinking in bridge applications and to present a reasonable way to carry out such analyses. Such research can lead to optimized structures, with regard to both costs and environmental impact, if implemented. Using LCCA and

LCA can also favour the establishment of new materials, which have potential of saving costs and decrease the environmental impact during the service life, in the bridge industry.

Both LCC and LCA are used in the construction industry today to some extent, but often separately. If the two approaches are combined it is possible to find a complete solution, considering both costs and environmental impact. A problematic issue is that it is difficult to express costs and emissions with a common unit. This implies that an integration of the two becomes subjective (Gervásio and da Silva, 2008).

In this thesis, different bridge designs have been studied by performing life-cycle cost analyses and life-cycle assessments. An existing bridge, owned by the Swedish transport administration, Trafikverket, has been evaluated and compared to alternative designs adapted to the same conditions. The bridge provided by Trafikverket is a conventional steel-concrete composite bridge, while in the alternative designs the bridge deck was substituted with a fibre reinforced polymer (FRP) deck or a steel sandwich deck. The purpose of the alternative designs is to evaluate the competitiveness of FRP and steel sandwich solutions, being relatively new for bridge applications.

This thesis is a part of work package 4, *Flexible construction techniques for new bridges*, of the European research project PANTURA, which focuses on developing and implementing innovative bridge designs (PANTURA, 2013). Work package 4 aims to implement new techniques and materials in bridge construction in order to reduce construction time and minimize interference with the surrounding environment.

1.1 Aim and objectives

The aim of the study is to contribute to the efforts which are made today to obtain more sustainable solutions in the bridge construction industry by reducing the costs and environmental impact of bridges. This is done, in this thesis, by an investigation of new materials and bridge concepts with regard to their competitiveness in a life-cycle perspective.

In conventional steel/concrete composite bridges, a large portion of the costs for maintenance activities and repair is attributed to the concrete deck. The objective is therefore to compare a conventional composite bridge using a concrete deck with steel bridges using fibre reinforced polymer deck and steel sandwich deck respectively. The comparison is made with regard to costs and environmental impact over the entire life span, from investment to demolition.

1.2 Method

To gain knowledge of the existing LCC and LCA methods a literature study was performed. With this background, suitable methods and tools were selected to conduct the LCC and LCA analyses. Moreover, different bridge techniques including FRP and steel sandwich decks were studied.

A preliminary design of the alternative bridge designs was made in order to find the material amounts and dimensions needed. Information about costs, maintenance, and emissions was retrieved from databases, such as Trafikverket's national database for bridge and tunnel management (BaTMan), literature, and by consulting the supervisors and other experienced professionals.

The LCCAs and LCAs were conducted separately using different software tools and hand calculations. The tools used were verified in order to achieve a reliable result. To identify critical parameters sensitivity analyses were conducted. The results were analysed separately and then interpreted together in order to reach a final conclusion with regard to both costs and environmental impact.

1.3 Scope

The study is limited to road bridges experiencing a high average daily traffic since great savings of costs and environmental impacts can be achieved in such conditions and since this is also the scope of PANTURA. The bridges compared are situated in Sweden, hence Swedish data and conditions are used. The design of the bridges is made in a preliminary manner in order to focus on the life-cycle analyses.

The study is focused on the economic and environmental aspects of life-cycle thinking while most of the social aspects are excluded since these are difficult to quantify. The only social aspects considered in this thesis are vehicle operation and driver delay. These are included as costs in the LCCA and emissions in the LCA. The study is also limited to using existing methods in the LCC and LCA analyses.

2 Life-cycle cost analysis

The term life-cycle cost (LCC), by definition, refers to the present value of the total cost of a given product or service over either the entire life span or a specified period of time (ETSI, 2012). The purpose of making life-cycle cost analyses is to enable a comparative economic assessment of all costs related to a project over a specified period of time. The comparison is only meaningful when the benefit of the alternatives is the same, meaning that, for example, two bridges with different capacities should not be compared (Safi, 2012).

The term LCC was first spread in the 1960's in the United States (Dhillon, 1989) and the first attempts to incorporate it in the construction industry were made in the mid 1980's (Gluch and Baumann, 2004). Since then, the theory of LCC has grown extensively and many research projects have been carried out to develop the LCC methodology in the construction sector. However, there is still a gap between theory and practice regarding the use of LCC among clients in the Swedish building sector (Sterner, 2000). According to a questionnaire sent out by Sterner (2000) to clients in the Swedish building sector, 35 out of the 53 responding clients used an LCC perspective, but not necessarily LCC calculations, when making investment decisions. The limited use of LCC was also indicated by an investigation commissioned by the European Commission in 2006, which showed that the practical application of LCC on real projects across Europe is not widespread (Davis Langdon Management Consulting, 2007b).

2.1 Life-cycle cost applications in different life-cycle phases

The life-cycle of a construction project typically consists of a design phase, a tendering phase, a construction phase, an operation and maintenance phase, and an end-of-life phase, as illustrated in Figure 2.1. Life-cycle cost analyses can be initiated in any of these stages (ETSI, 2012) but is most often implemented in the design phase in order to compare different design alternatives to find the most cost-effective solution (Sterner, 2000). This is also the phase with the largest saving potential (Safi, 2012) since the cost for making changes increases rapidly after the design has been chosen (Sterner, 2000).

Figure 2.1. Life-cycle phases for LCC

LCCA can also be used beneficially in the tendering phase where the concept of the lowest bid is often used to choose the contractor (Safi, 2012). Traditionally, the lowest bid refers to the lowest initial investment cost, but more cost-efficient projects can be attained if the lowest bid refers to the life-cycle costs.

When the product is in service, LCC can be implemented to choose among different repair alternatives or to assist when making decisions whether to repair or replace it (Safi, 2012). Finally, LCC can be used at the end-of-life stage to choose the optimal demolition strategy in a cost perspective.

2.2 Life-cycle costs of bridges

The costs included in life-cycle costing can be divided into agency costs, user costs and society costs with further subdivisions as shown in Figure 2.2 (ETSI, 2012).

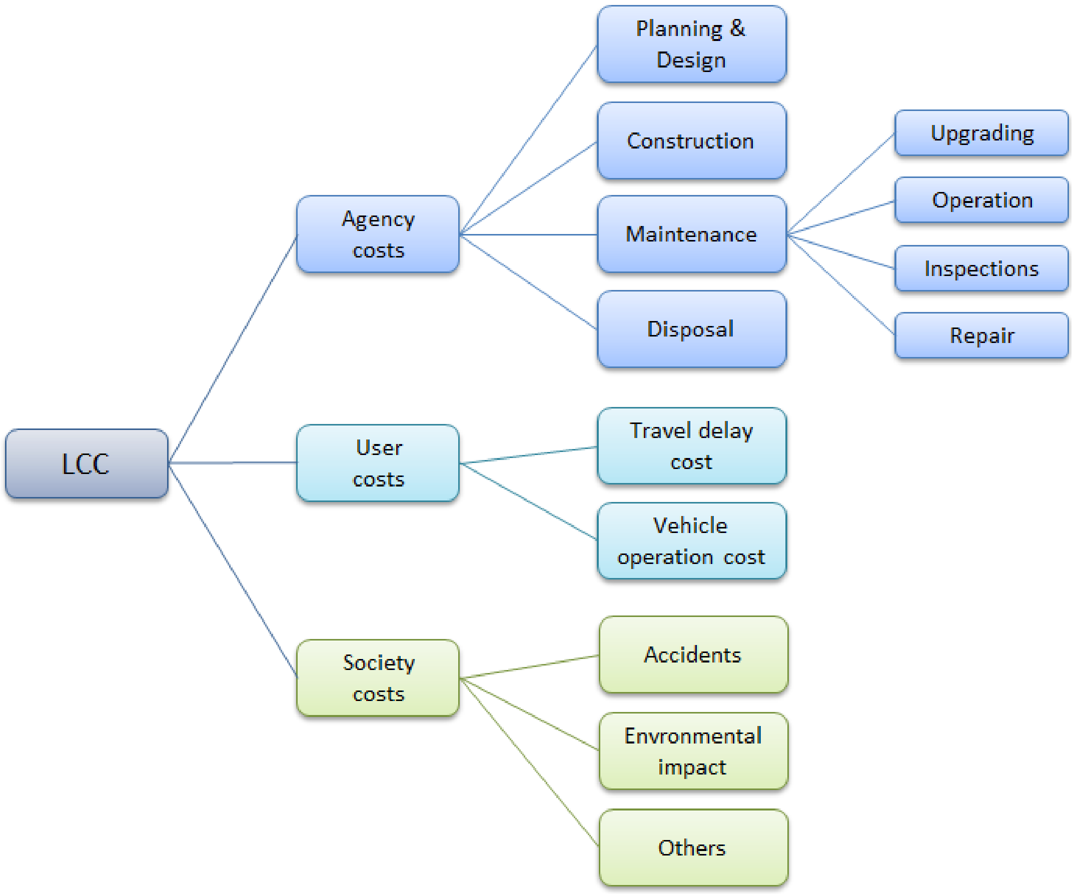


Figure 2.2. LCC cost scheme for a bridge (ETSI, 2012) (reproduced by the authors)

Many of the costs displayed in Figure 2.2 occur at different times during the life-cycle, and to be able to summarize them into one total cost it is necessary to compare past, present and future costs on a common basis. This is usually done with the 'Net Present Value'(NPV) method, which is based on the principle that it is more valuable to have money at hand today than at a future date (Davis Langdon Management Consulting, 2007a). The NPV method uses a discount rate to transfer all future costs to today's value, as expressed in Equation (1) (Safi, 2012).

$$NPV = \sum_{n=0}^{L} \frac{C_n}{(1+r)^n} \tag{1}$$

where

NPV is the life-cycle cost expressed as a present value

n is the year when the cost occurs

C_n is the sum of all cash flows in year n

r is the discount rate

L is the service life span

The use of the NPV method is only appropriate when the life spans of the compared alternatives are the same (Safi, 2012). If this is not the case, the 'Equivalent Annual Cost' technique can be used instead. This technique calculates the cost per year of owning and operating an asset.

When it comes to discount rates, a distinction can be made between 'real' discount rate and 'nominal' discount rate (Davis Langdon Management Consulting, 2007a). When using 'nominal' discount rate, the future cost of a product is predicted including inflation. On the other hand, 'real' discount rate excludes the effect of inflation, which means that future costs are estimated as the 'real' present day prices. Since the inflation is difficult to predict in the long term, 'real' discount rates are recommended to use in LCC calculations for long term investments such as bridges.

The value of the discount rate depends on the purpose of the analysis and who is conducting it (Davis Langdon Management Consulting, 2007a). Using a low discount rate means that a larger consideration for future costs is made and it is often used by public authorities (typically 2-5%, 'real'). A high discount rate can be used when the risks of making an investment are larger and the future costs are not considered as important. This tends to be favoured by private investors (typically 2-14%, 'real') (Davis Langdon Management Consulting, 2007b). In Sweden, Trafikverket recommends using a 'real' discount rate of 3,5% for infrastructure projects (SIKA, 2005, Trafikverket, 2012c). The value chosen for the discount rate can have a great impact on the outcome of an LCC analysis.

2.2.1 Agency costs

Agency costs, also referred to as direct costs, are the expenses of the owner of the asset such as investment costs, remedial action costs and end of life management costs (Safi, 2012). The agency costs can be calculated according to Equation (2) (ETSI, 2012). Note that in this equation, the end-of-life costs are included in the term $C_{MR\&R}$.

$$C_{agency} = C_{acquisition} + C_{MR\&R} \tag{2}$$

where

$C_{acquisition}$ is the cost for purchasing, construction and installation

$C_{MR\&R}$ is the cost for inspections, operation, maintenance, repair and disposal

C_{agency} are the expenses of the owner

When using the NPV method to calculate the agency cost, it is necessary to know the time and cost of every maintenance activity. These parameters are difficult to predict and the lack of reliable data is one of the greatest constraints for conducting life-cycle cost analyses (Sterner, 2000). Concerning bridges, assumptions regarding the operation and maintenance costs are often based on historical data from actual bridge inspections and repairs. In Sweden such information can be retrieved from the database Bridge and Tunnel Management system, BaTMan (Safi, 2012). BaTMan is provided by Trafikverket and is an online database containing information about bridges in Sweden since 1944.

2.2.2 User costs

When LCC analysis is used for bridge applications the user costs are typically the indirect costs for drivers, vehicles and transported goods on the bridge (ETSI, 2012). The user costs arise due to traffic disruptions when construction work is carried out on the bridge, leading to an increased vehicle trip time, discomfort and increased risks. Here, the user costs are divided into travel delay cost and vehicle operation cost. The travel delay cost takes into account the additional time that drivers spend in traffic due to construction work, which leads to lost working hours for the user. The travel delay cost can be calculated according to Equation (3) (Safi, 2012).

$$C_{TDC} = T * ADT_t * N_t * (r_T w_T + (1 - r_T) w_p) \tag{3}$$

where

C_{TDC} is the travel delay cost

T is the travel time delay for one vehicle (hours)

ADT_t is the average daily traffic on the bridge at time t

N_t is the number of days of road work at time t

r_T is the percentage of trucks of the total ADT

w_T is the hourly cost for one truck

w_p is the hourly cost for one passenger car

The travel time delay T in Equation 3 can be calculated in different ways depending on whether the delay is caused by a speed reduction, traffic light regulations or traffic diversions (ETSI, 2012). Information about the average daily traffic (ADT) and the percentage of trucks can usually be retrieved from Trafikverket. The hourly cost of a passenger car and a truck is an estimation of the time value and varies from country to country.

Another user cost is the vehicle operation cost which accounts for the additional time that the vehicle needs to be operated due to the traffic disturbances (Safi, 2012). It includes costs for fuel, engine oil, maintenance etc. and is calculated according to Equation 4. In the work of Safi (2012), the vehicle operation cost and the traffic delay cost were combined giving a total cost of 167 SEK/h for passenger cars and 347 SEK/h for trucks.

$$C_{VOC} = T * ADT_t * N_t * \left(r_T O_T + (1 - r_T) O_p\right) \tag{4}$$

where

O_T is the average hourly operating cost for one truck including its goods operation

O_p is the average hourly operating cost for one passenger car

C_{VOC} is the vehicle operation cost

For, further explanations, see Equation (3)

2.2.3 Society cost

Traffic accidents, causing costs in terms of health-care and deaths, are an example of society costs arising during the life time of a bridge (ETSI, 2012). In cases when the purpose of the LCC analysis is to compare different bridges, this cost only needs to be included if the alternatives have different probabilities for accidents. In that case, Equation (5) is used. According to Trafikverket, the cost for the society per fatality is around SEK 24 million and the cost per serious injury is about SEK 4.5 million (Trafikverket, 2012b).

$$C_{ACC} = \sum_{t=0}^{L} ADT_t * N_t * (A_a - A_n) * [(C_F * P_F) + (C_I * P_I)] \frac{1}{(1+r)^t} \quad (5)$$

where

A_n is the bridge accident rate during normal conditions

A_a is the bridge accident rate during the work activities

C_{ACC} is the accident cost

C_F is the average cost per fatality for the society

C_I is the average cost per serious injury accident for the society

P_F is the average number of persons killed in bridge-related accidents

P_I is the average number of persons injured (not killed) in bridge-related accidents

It is also possible to include costs that arise if the bridge would fail, but since the risk for failure is considered to be very small this is usually omitted in the analysis (ETSI, 2012).

Other society costs are costs related to the environmental damage caused by the bridge, for example due to emissions and resource consumption. One way to consider environmental impact as a cost is by multiplying the cost of the used material by a factor in order to account for the embodied energy from manufacturing and transportation (ETSI, 2012). There are many views upon how and if environmental costs should be included in LCCA and this is discussed further in Chapter 4.

Construction work can also lead to indirect costs for the society, for example in terms of loss of income for retailers whose customers are bothered by the construction activities (Ehlen, 1999). However, such costs are seldom considered in the LCCA since they are difficult to define (Gervásio and da Silva, 2008).

2.3 Methodology: how to perform a life-cycle cost analysis

In 2006, the European Commission started a project aiming to improve the competitiveness in the construction industry by developing a common methodology for life-cycle cost analysis at European level (Davis Langdon Management Consulting, 2007a). The project resulted in a methodological framework to enhance a common and consistent application of LCC in European countries. The methodology is compatible with the international standard ISO15686, Part 5 (Davis Langdon Management Consulting, 2007b) and consists of the following 15 steps which are further explained in the next subchapters (Davis Langdon Management Consulting, 2007a).

1. Identify the main purpose of the LCC analysis
2. Identify the initial scope of the analysis
3. Identify the extent to which sustainability analysis relates to LCC
4. Identify the period of analysis and the methods of economic evaluation
5. Identify the need for additional analyses (risk/uncertainty and sensitivity analyses)
6. Identify project and asset requirements
7. Identify options to be included in the LCC exercise and cost items to be considered
8. Assemble cost and time data to be used in the LCC analysis
9. Verify values of financial parameters and period of analysis
10. Review risk strategy and carry out preliminary uncertainty/ risk analysis
11. Perform required economic evaluation
12. Carry out detailed risk/uncertainty analysis (if required)
13. Carry out sensitivity analyses (if required)
14. Interpret and present initial results in required format
15. Present final results and prepare a final report

2.3.1 Identification of parameters and analysis requirements

The first seven steps are about identifying the parameters and data needed to conduct the LCC analysis. In order to do this it is necessary to start by stating the purpose of the analysis and have an idea of how the analysis should be implemented and what it will result in (Davis Langdon Management Consulting, 2007a).

The system boundaries should be defined, i.e. what stages in the life-cycle should be considered and furthermore which costs are included within these stages (Davis Langdon Management Consulting, 2007a). It is also necessary to determine how to handle the environmental impact as well as deciding what calculation tools should be used.

When a physical description of the asset and its components has been made it is possible to describe the system boundaries in a more detailed manner. This covers determining the life span and making a maintenance activity plan.

One important step is to identify the risks and uncertainties in the analysis. In order to get as reliable results as possible these should be handled in some way. One tool that can be used is a sensitivity analysis where an uncertain parameter is changed to see how the overall result is affected.

2.3.2 Assembly of cost and time data

When all parameters needed for the LCCA have been identified, each cost should be quantified. In the early stage of the design phase, limited data is available and most information about the costs and when the costs occur is obtained from the client's own records or published national data sets (Davis Langdon Management Consulting, 2007a). There are data sets available in different countries. However they are not always comparable since there is no universal standard for what specific costs should include.

As the level of detail of the design increases, more reliable data on costs and time intervals can be retrieved from the manufacturers. Another important aspect to consider when making a comparative analysis is to make the same assumptions and simplifications for all the studied alternatives.

2.3.3 Carrying out the analysis

The LCC analysis is often carried out using commercial or in-house software which incorporate tools to make NPV calculations or use other economic evaluation techniques (Davis Langdon Management Consulting, 2007a). A few available software tools are presented in Chapter 2.4. However, most calculations in an LCC analysis are not complicated and can be done manually using a simple computational tool like Microsoft Excel and the equations presented earlier in Chapter 2.

If required, a detailed risk or uncertainty analysis such as a Monte Carlo simulation can be carried out at this stage in order to evaluate the credibility of the results of the LCC analysis. The needed input data in a Monte Carlo simulation are the probabilities of each uncertain parameter and the type of probability distribution (e.g. uniform, triangular or discrete) that should be used (Davis Langdon Management Consulting, 2007a). The output can be illustrated in a graph showing for example the probability distribution of different life-cycle costs, as shown in Figure 2.3.

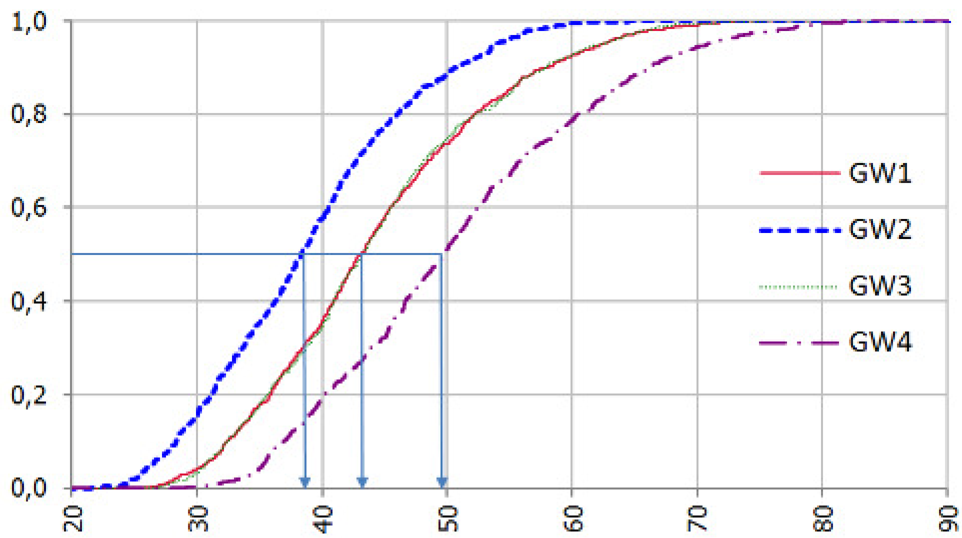


Figure 2.3. Example of a graph showing the probability of different outcomes for four different alternatives. The arrows indicate the 50-percentile probability for each alternative (Gervásio and da Silva, 2013).

As mentioned in Chapter 2.3.1, another option for handling uncertainties is to conduct a sensitivity analysis. This is the most widely used deterministic risk analysis method in project risk management since it is easy to perform and interpret (Davis Langdon Management Consulting, 2007a). The procedure consists in iteratively increasing or decreasing the value of an individual input parameter so that a risk-adjusted life-cycle cost can be presented.

2.3.4 Interpreting and reporting results

Since an LCC analysis inevitably contains simplifications and assumptions, the results should be presented in a way that informs the reader about the uncertainties and limitations of the findings (Davis Langdon Management Consulting, 2007a). The common methodology suggests keeping the following points in mind when interpreting the results:

- LCC is not a precise science and the reliability of the outcomes should be regarded as, at best, ‘reasonable’
- LCC outputs can never be more accurate than the inputs, in particular the estimates and assumptions made regarding both time and cost
- The accuracy of results is difficult to measure as the variances obtained by statistical methods are often large
- Relevant data can be both difficult and expensive to acquire, particularly regarding the operation and maintenance phase in the life-cycle

When making life-cycle cost analyses for bridges, special considerations regarding the accuracy of the analysis should be made since the analysis period is often very long. This entails higher risks because inflation, future need and use of the bridge, and deterioration are long term effects which are difficult to predict (Davis Langdon Management Consulting, 2007a). Moreover, the impact of the chosen discount rate increases the longer the analysis period is.

2.4 Life-cycle cost software tools

There are several software tools available for computing LCC analyses; some are specialized on a certain product whereas others are more general. Two software tools which are appropriate for conducting LCCA on bridges are BridgeLCC and Bridge-stand-alone-LCC (ETSI, 2012, Larsson and Nilsson, 2011). BridgeLCC was developed by the National Institute of Standards and Technology (NIST) in the United States. This software has not been updated since 2003 and in an investigation done by Larsson and Nilsson (2011) the input factors were sometimes scaled up by a factor of 1000 without reason, making the output data unreliable. The other LCC software, *Bridge-stand-alone-LCC*, is user friendly and has been updated continuously up to date. Bridge-stand-alone-LCC was developed in a Finnish led research project called ETSI, which stands for *Elinkaareltaan tarkoituksenmukainen silta*, meaning bridge life-cycle optimisation (Hammervold et al., 2009).

3 Life-cycle assessment

Life-cycle assessment (LCA) is an approach which enables quantifying of the environmental impact of a product or process over its entire life span (Lippiatt, 2009, Du, 2012). In an infrastructure project, LCA takes into account all environmental impacts of an asset throughout its life time including raw material acquisition, construction, maintenance, transport, and disposal (Du, 2012, Gervásio and da Silva, 2013). The results can be used as support in a decision-making process, for example when different alternative designs are suggested in a project.

One of the advantages of LCA is that it is multi-dimensional and takes several different environmental impacts into consideration. In the construction sector LCA is relatively well-known and it is the only environmental analysis regulated by an international standard, ISO 14040 (Davis Langdon Management Consulting, 2007a). On the other hand the long life span can be a drawback when using LCA to evaluate bridges, life since the analysis will be less precise over time (Gervásio and da Silva, 2013). There are also a number of uncertainties in the process which limit the use of LCA, these are addressed in Chapter 3.1.5 (Du, 2012, Gervásio and da Silva, 2013).

LCA was first developed during the 1960's and 1970's (Russell et al., 2005). One of the first studies on LCA was presented at the World Energy Conference in 1963 by Harold Smith (University of Bath, 2013, Ho, 2011). In 1969, Coca-Cola was the first company to perform an LCA, where different containers were evaluated (Du, 2012, University of Bath, 2013, Ho, 2011). During the 1970's the first software handling LCA was created and the development of the method was driven by increased awareness of fossil depletion and the oil crisis.

In the early age of LCAs, there was no standard procedure on how to perform an LCA which often led to contradictory results (Russell et al., 2005, Du, 2012, University of Bath, 2013). During the 1990's, development of an international standard was initiated by both the Society of Environmental Toxicology and Chemistry (SETAC) and the International Organization of Standardization (ISO). Their work resulted in the extensive ISO 14040 series, which are the European standards and the SETAC Code of practice. .

The interest in LCA has grown rapidly and it is used in many different industries. In recent years, research on LCA in the bridge industry has increased (Hammervold et al., 2009, Du, 2012). However, the use of LCA in the construction industry has been limited and its practical use needs to be evaluated (Landolfo et al., 2011).

3.1 Methodology: how to perform a life-cycle assessment

The standardized LCA is divided into four phases; goal and scope definition, inventory analysis, impact assessment and interpretation, see Figure 3.1 (ISO, 2006). These steps are performed in sequence but it is important to remember that an LCA is an iterative process. If new information is found during the process the study should be revised. It is also important to keep the LCA as transparent and comprehensive as possible due to its complex nature.

The four phases, all according to ISO 14040 standards unless stated otherwise, are described below. The presented equations are according to the ETSI project (Hammervold et al., 2009). In the end of the chapter, the limitations and uncertainties included in an LCA are addressed.

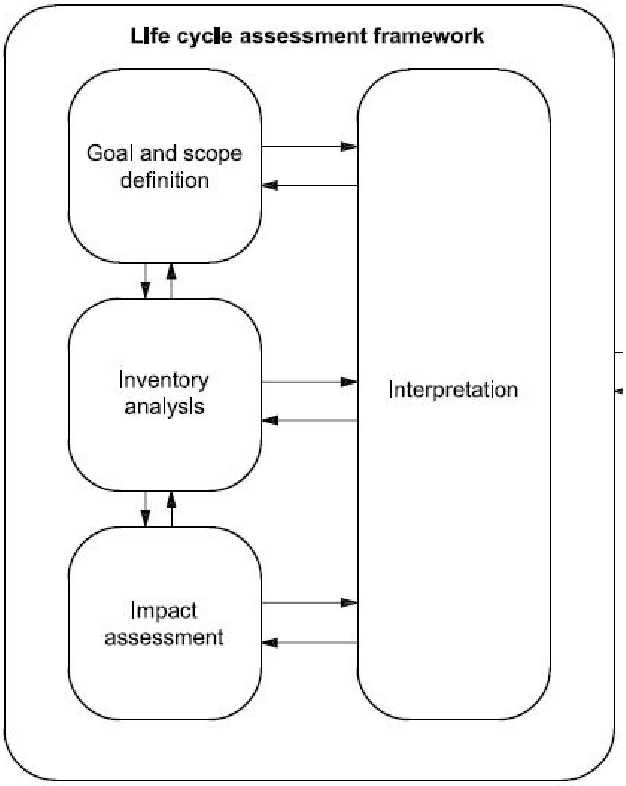


Figure 3.1. The four phases of LCA as described in ISO 14040 (ISO, 2006) (edited by the authors).

3.1.1 Phase 1: Goal and scope definition

In the first step, the goal and scope of the study are defined and this is the foundation of the entire LCA study (Du, 2012). The goal should state the intended application of, the intended audience for, and the reason to perform the study. The scope should include the function of the studied product system (the functional unit), the system boundary, selected environmental impact categories and methodologies, limitations, and assumptions among else (Hammervold et al., 2009, Baumann and Tillman, 2004). It is important that the goal and scope are clear to ensure that the finished LCA will be useful as a decision support. A clear definition of the functional unit is essential if a comparison between different alternatives should be performed (Du, 2012).

As part of the scope, a system boundary is chosen. The system boundary is an important part as it defines which life-cycle stages, inputs and outputs should be included in the LCA. To make a credible LCA it is important that all flows giving a substantial contribution to the environmental impact are included. Different approaches on which life-cycle stages should be included are possible, for example 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. Thus, the system boundary defines how detailed and extensive the LCA will be (Lippiatt, 2009).

3.1.2 Phase 2: Life-cycle inventory

Life-cycle inventory (LCI) is the second phase of the LCA and includes data collection and quantification of data. Material and energy input, and output in terms of emissions should be defined and the corresponding environmental stressors, such as pollutants, should be identified. This can be done in a flow chart to make the process easy to survey (Hammervold et al., 2009, Du, 2012). Then, data can be collected, typically using LCA databases such as ecoinvent or Ecobalance LCA database, suppliers, reports, and software tools (Du, 2012, Hammervold et al., 2009, Lippiatt, 2009). The collected data can then be used to quantify the flows of emissions, materials and energy. For example, if a bridge is analysed traffic disruptions will occur during construction and maintenance. Traffic disruptions cause extra emissions of fumes, and this is part of the output in the LCI. The emissions, among else, give flows of greenhouse gases and SO_2, which are stressors, and should be quantified as in Equation (6). 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).

$$e_{ij} = x_i * f_{ij} \tag{6}$$

where

e_{ij} are the emissions of the stressor j for the total consumption of input parameter i

x_i is the consumption of input parameter i

f_{ij} are the emissions of stressor j per unit of input parameter i

3.1.3 Phase 3: Life-cycle impact assessment

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 else 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, indicators and characterization models, classification and characterization while the optional are normalization, grouping and weighting. The general procedure is shown in Figure 3.2, while an example can be seen in Figure 3.3.A description of each element in an LCIA follows below. The LCIA is usually the phase of LCA claiming most time (Du, 2012).

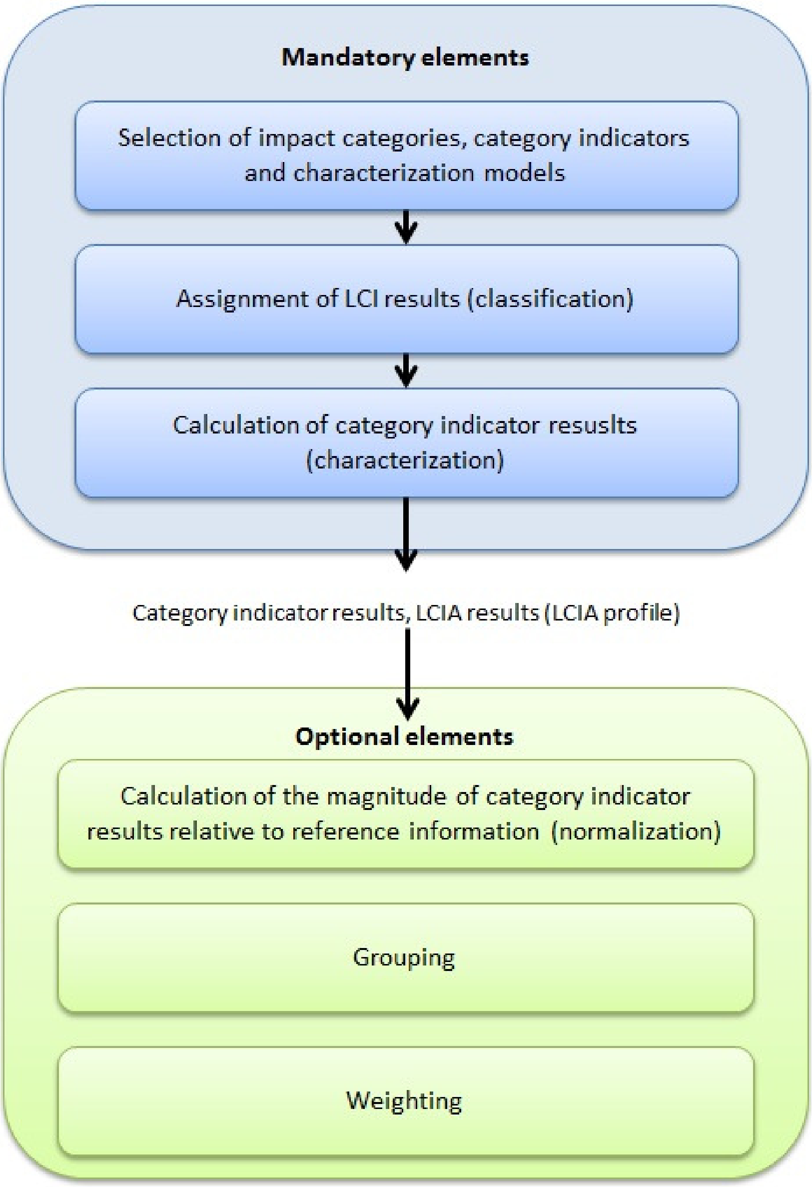


Figure 3.2. The six elements of LCIA (ISO, 2006) (reproduced by the authors)

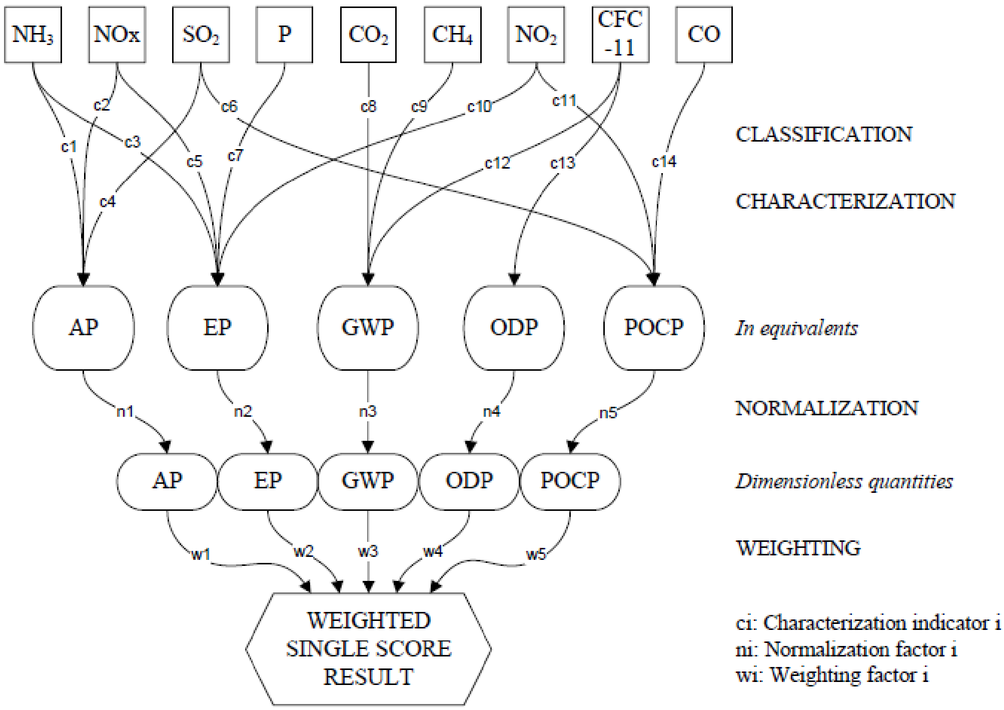


Figure 3.3. Example of an LCIA (Hammervold et al., 2009)

Selection of impact categories, category indicators and characterization models

A set of impact categories is selected, defining which environmental impacts will be taken into account. There are both midpoint and endpoint level impact categories; midpoint level categories are problem-oriented while endpoint level categories are damage-oriented (Du, 2012). For instance, human toxicity is an impact on midpoint level while human life expectancy is an endpoint level impact. There are no standards defining which categories should be included, but impact categories at midpoint level that occur frequently are; global warming potential (GWP), acidification potential (AP), eutrophication potential (EP), abiotic depletion potential (ADP), ozone depletion potential (ODP), ecotoxicity (ETC), and human toxicity (HTC) (Du, 2012, Hammervold et al., 2009, Baumann and Tillman, 2004, Gervásio and da Silva, 2008). These choices could also be made in the first phase as part of the scope definition (Hammervold et al., 2009).

Classification

Classification means that all stressor flows are connected 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. If a stressor contributes to more than one impact category it should be connected to all of them, in this case the contribution could either be divided between the impact categories or fully allocated to both of them (Du, 2012).

Characterization

The next element is characterization, where all relevant flows for an impact are transferred into a common unit, see Equation 7. This means that 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).

$$d_k = \sum_{i=1,j=1}^{i,j} \left(e_{ij} * c_{jk}\right) \tag{7}$$

where

d_k is the total potential impacts in impact category k, expressed in equivalents

e_{ij} are the emissions of the stressor j for the total consumption of input parameter i

c_{jk} is the characterization indicator for stressor j to impact category k

Normalization

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). A possible approach is to relate to a smaller area, like a country, for impacts that affect the local environment, such as acidification (Gervásio and da Silva, 2013). Other impacts, which act on a global level, can be related to continents or the entire globe. For example, the GWP of an alternative is expressed in CO_2-equivalents. The CO_2-equivalents emissions of the alternative are then expressed as part of the total CO_2-equivalents emission per capita globally, see Equation (8).

$$m_k = d_k * n_k \tag{8}$$

where

m_k is the normalized potential impacts for category k

d_k is the total potential impacts in impact category k, expressed in equivalents

n_k is the normalization factor for category k

Grouping

In the grouping step the results of the impact categories are grouped to facilitate the interpretation of the results (Baumann and Tillman, 2004, Du, 2012). The categories can be sorted either by specific properties or priorities.

Weighting

Weighting is the last step of the LCIA and aims to make the results of LCAs for different alternatives comparable. It is only meaningful to do a weighting if the intension is to perform an overall comparison between different options (Gervásio and da Silva, 2013). The point of the weighting is to rate different environmental impacts by their importance for the overall environmental performance (Gervásio and da Silva, 2008, Lippiatt, 2009). The weighting can be based on political goals, monetisation, limitations of emissions etc. (Du, 2012, Hammervold et al., 2009). A number of weighting sets have been developed and can be used, such as US-EPA, Harvard, BEES, and EDIP, these can be found in (Hammervold et al., 2009, Lippiatt, 2009). In the Harvard set, impacts have been evaluated with regard to both future and current consequences, with future consequences given a larger influence (Gervásio and da Silva, 2008). Another possibility is to use multi-attribute decision analysis (MADA) which is a tool that can be used to evaluate problems where conflicting interests must be considered (Gervásio and da Silva, 2012). The use of MADA can decrease uncertainties and subjectivity in the weighting procedure.

$$v = \sum_{k=1}^{k} (m_k * w_k) \tag{9}$$

where

w_k is the weighting factor for impact category *k*

m_k is the normalized potential impacts for category *k*

v is the total weighted result, sum of all impact categories

Table 3.1 Weighting factors for normalised LCIA results,(Hammervold et al., 2009)

	Fossil depletion	Acidification	Eutrophication	Global warming	Ozone depletion
US-EPA	5	5	5	16	5
Harvard	7	9	9	11	11
BEES default	9	9	9	9	8
EDIP	0	1.3	1.2	1.3	23

3.1.4 Phase 4: Interpretation

Interpretation is the last phase of the LCA, where the results of the previous phases are evaluated in relation to the goal and scope stated in the beginning of the process (ISO, 2006). During the interpretation, all uncertainties and limitations of the study should be clarified so that credible conclusions can be drawn (Du, 2012).

How the interpretation is made depends on if the optional elements (normalization, grouping, and weighting) were included in the LCIA. If they were not included, the environmental impact can be compared category by category. This approach can make an overall comparison of the alternatives difficult, since one alternative seldom outperforms another in all impact categories and since different categories are expressed in different units (Lippiatt, 2009). If, on the other hand, the optional elements are included in the LCIA one total score is achieved for each alternative, making them comparable. However, these scores will always be subjective to a certain extent due to the choices made in the LCIA. According to ISO 14040 recommendations, a weighted total score should not be used as decision-support since there is no scientific base for the weighting. Therefore, it is necessary to have access to the results for each impact category as well to keep transparency (Hammervold et al., 2009).

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 (ISO, 2006, Hammervold et al., 2009). It is also important to make a sensitivity analysis of the gained results to be able to see how the choices made during the assessment affect the outcome (ETSI, 2012, Gervásio and da Silva, 2013). Furthermore, a sensitivity analysis shows which parameters influence the results the most (ETSI, 2012). Uncertainties in such influential parameters should be considered when drawing conclusions or making recommendations based on the LCA.

3.1.5 Uncertainties and limitations of the life-cycle assessment

There are a number of limitations and uncertainties in a performed LCA and it is important to be aware of the effects they have on the results. Uncertainties and limitations exist in all the four phases and are introduced with input data and choices of models and scenarios. The most influential uncertainties are described below (Gervásio and da Silva, 2013).

During the life-cycle inventory (LCI) the largest obstacle lies in the collection of data. The amount of data needed is often extensive and large resources are necessary. There is also a lack of relevant data in databases. If the actual production process differs from the one in the database, the given data will not be entirely correct (Du, 2012). Other sources of inaccuracy are outdated or incomplete data, or data that does not apply to the local environment (Gervásio and da Silva, 2013). Therefore, it is desirable to use specific data provided by manufacturers and builders if available. It is easier to estimate flows for steady, long-term and wide-spread processes, which makes an LCA of such a process more precise than an LCA of a transient and local process (Hammervold et al., 2009). Regardless of how data is collected, there is a substantial risk of including uncertainties in the assessment (Gervásio and da Silva, 2013).

In the life-cycle impact assessment (LCIA) a number of choices, regarding for example impact categories, are made by the decision-maker. Together with the choice of parameters, models and methodology this introduces a number of uncertainties and subjectivity to the LCA (Gervásio and da Silva, 2013, Du, 2012, ISO, 2006, ETSI, 2012). The LCIA is also limited by the lack of standard procedure and guidelines for the process (Du, 2012). For example, LCIAs are not necessarily comparable since they can include different impact categories or have completed different steps.

During the normalization, two main sources of uncertainties are lack of emission data and the modelling in the characterization step (Gervásio and da Silva, 2013). Moreover, if a weighting is performed, there is no standard since weighting always must rate economic, social, environmental, and political interests against each other and these are questions where there are generally no common agreement (Du, 2012). Aiming to avoid subjectivity, it is important to clearly state all the assumptions, in order to reach transparent and reliable conclusions and recommendations. Likewise, it is necessary to check sensitivity, completeness and consistency of the LCA (ETSI, 2012). Gervasio and da Silva (2013) especially points out the need for sensitivity analysis of the weighting, since this is the step where most subjective choices are made.

There are also limitations concerning the scientific base for LCIA. According to Hammervold, 2009, SETAC claims that the toxicity impact categories must be further developed and that they do not meet scientific requirements. Furthermore, all environmental impacts are added up during the LCIA without considering possible interactions between chemicals that could increase or decrease their effect (Hammervold et al., 2009). Moreover, differences in sensitivity to contamination between different areas are neglected.

To achieve objective results of an LCA, Gervasio and da Silva (2013) suggest the use of probabilistic analysis methods that takes many dimensions into consideration by performing Monte Carlo and bootstrap simulations. Using this approach, several uncertainties can be considered in the analysis. Thus, such a study would not single out one alternative with a good score as better than the others but the probabilities of each alternative being the best one.

3.2 Life-cycle assessment software tools

There are a number of LCA software tools available on the market. However, when it comes to software tools specializing in the building sector the number is limited and very few are bridge specific (Du, 2012). The Swedish Industrial Design Foundation (SVID) recommends the well-known software SimaPro for thorough LCAs or Eco-it for simpler LCA evaluations (Hållbarhetsguiden, 2013). Some of the software tools specifically aimed at the building sector are BEES, ATHENA Impact Estimator, and

ETSI BridgeLCA (Du, 2012). Out of these, only ETSI BridgeLCA is specifically aimed at bridges.

SimaPro is the most common LCA software internationally (Hållbarhetsguiden, 2013), and can be used to make extensive LCAs. SimaPro follows the standards of ISO 14040 and uses a number of databases, among else, the ecoinvent database (Du, 2012, Miljögiraff, 2013). ecoinvent claims to be a world leading database with large amounts of data available (Ecoinvent centre, 2013). SimaPro also uses Monte Carlo simulations in their life-cycle assessments (PRé sustainability, 2013).

Eco-it is a simpler tool, developed by the same company as SimaPro, but easier to handle and less-time-consuming (Hållbarhetsguiden, 2013, Prè sustainability, 2013). It is useful when the aim is to quickly get an idea of which stage in the product's life has the greatest environmental impact.

Developed by NIST in USA, BEES is a tool performing LCAs on building materials and products (The National Institute of Standards and Technology, 2011). BEES follows the international ISO standards and uses MADA analysis in their assessments.

The ATHENA Impact Estimator is developed by Athena Sustainable Materials Institute, and is a software that enables comparison and evaluation of up to five different design solutions (Athena Sustainable Materials Institute, 2013, DG Joint Research Centre, 2012). The Impact Estimator was originally a tool for buildings but a version for highways is under development (Athena Sustainable Materials Institute, 2013). The Athena Sustainable Materials Institute tools are based on ISO 14040 and use the US Life-cycle Inventory database.

ETSI BridgeLCA is a tool developed as part of the ETSI-project and specialized for LCA on bridges. BridgeLCA uses imported data and impacts from SimaPro and, in extension, the ecoinvent database (Hammervold 2009).

openLCA is an open source LCA tool initiated and run by GreenDelta (GreenDelta GmbH, 2012). It can be connected to different databases and impact assessment methods as desired by the user.

4 Integration of LCC and LCA

LCC and LCA analyses are valuable tools for aiding decision-making in order to achieve cost efficient or environmental friendly infrastructure projects respectively. The main motive for integrating LCC and LCA analysis is that the LCA is not expressed in monetary value and therefore tends to be neglected by the business managers (Shapiro, 2001). If the two are integrated, there is a potential to enhance a sustainable development in the construction industry (Rebitzer and Seuring, 2003).

Many researchers have seen the need to merge environmental and economic performance and have addressed this by proposing several methods for integration of the two (Gu et al., 2009, Lippiatt, 2009, Shapiro, 2001, Deng et al., 2008, Norris, 2001, Gervásio and da Silva, 2008, Gervásio and da Silva, 2013, Kendall et al., 2008, Palousis et al., 2008). However, according to an investigation lead by the European Commission in 2006, experts in both LCC and LCA fields claim that it is not generally feasible to merge the available LCC tools with LCA methods (Davis Langdon Management Consulting, 2007b).

Both LCC and LCA are relatively new approaches in the production industry and the idea of integrating the two has therefore not been relevant until recently. Thus, most of the existing methodologies that are actually used cover either LCC or LCA individually (Palousis et al., 2008). This separation has limited the influence and relevance of LCA in decision-making (Norris, 2001, Deng et al., 2008). In the scientific papers which treat the issue of integrating LCC and LCA, there are few case studies and the methodologies proposed for integration seem to still be on a theoretical level.

4.1 Existing integration approaches

There are several possible ways to use LCA together with LCC. One option is to use LCC and LCA as evaluation criteria in the decision-making process alongside with other criteria such as functionality, aesthetics etc. (Davis Langdon Management Consulting, 2007a). This requires a subjective weighting of the different criteria.

In an integration approach proposed by Lippiatt (2009), an LCCA and an LCA analysis are first performed separately and a score for the winning alternative of each analysis is obtained. The scores are then rescaled on a scale from 0 to 100 by dividing the score of the winning alternative by the sum of the score of all alternatives included in the analysis. The environmental and economic performances are then combined using the American Society for Technology and Materials (ASTM) standard for multiattribute decision analysis (MADA). In practice, this means that an overall score is obtained by weighting the environmental and economic performances relatively to each other. This integration method is used in the software tool BEES developed by Lippiatt. It has also been used in a study by Gervasio and da Silva (2008) where two different highway bridge designs were compared with respect to economy and environmental impact.

In a later study, Gervasio and da Silva (2012) used the same methodology as in 2008 but extended it to include uncertainties in the analysis. This was done by a probabilistic analysis which does not give a single best alternative but instead presents the probability for each alternative to become the winning solution.

Another possibility is to incorporate environmental impact into the LCC by assigning them a monetary value (Davis Langdon Management Consulting, 2007a). For some environmental impacts there is a standardized cost. However, the methodology of setting a monetary value on environmental impacts is controversial and questioned by many environmental experts (Davis Langdon Management Consulting, 2007b). For example, putting a prize on endangered species habitat or ultimately human life is difficult both from a practical and ethical point of view (Lippiatt, 2009, Shapiro, 2001).

One method that puts a price on emissions was developed by Kendall, Keoleian and Helfand (2008) using the output of the LCA as input in the LCCA. Due to the uncertainties regarding the costs of pollution, a Monte Carlo simulation is carried out on these costs.

One tool which is used to supplement environmental information with cost information is environmental accounting (Shapiro, 2001). This tool accounts for material flows and costs, including environmental cost, by assigning a monetary value to the external costs. The costs can be estimated *directly* based on for example lost revenue, or *indirectly*. There are two methods for making an *indirect* assessment of the cost. Either the estimation is based on surveys or interviews which investigates the willingness to pay for the preserving of a natural resource, clean freshwater etc. The other option is to estimate the cost by using the existing market behaviour.

Palousis, Luong and Abhary (2008) presented an integrated LCA/LCC framework for assessing product sustainability risk. This approach puts a price on the environmental effects in a different way than the previous examples. LCA and LCC analyses are first carried out separately and are followed by a sustainability risk assessment. This risk assessment identifies possible risks and when in the life-cycle they would occur. Moreover, it evaluates the probability and severity of each identified risk. The LCC is then updated so that each cost get a percentage increase corresponding to the probability and severity of the risks assigned to that cost.

LCC and LCA can also be used in sequence by first carrying out an LCCA in order to pick out economically good alternatives and then doing an LCA analysis on those alternatives, or vice versa (Davis Langdon Management Consulting, 2007a). This way, an alternative with acceptable performance regarding both cost and environmental impact can be chosen.

5 Case study of different bridge designs

The case study was carried out to evaluate the competitiveness of three bridge designs with fibre reinforced polymer (FRP) decks and one design with a steel sandwich deck, compared to a conventional bridge built in Sweden. The designs were compared with regard to their environmental impact and costs during the service life by life-cycle assessment and life-cycle cost analysis respectively. The result of the study is meant to indicate if these innovative bridge designs are a competitive for building new bridges in the specific situation considered in this study.

5.1 Introduction to fibre reinforced polymers

FRP is a relatively new material in the bridge industry, which has been used in bridge construction in for example the United States, the United Kingdom and Germany (Bakis et al., 2002, Kendall, 2006). One of the main advantages is its light weight and its high strength/weight ratio (Bakis et al., 2002, Karbhari and Zhao, 2000). FRP is also a durable material with high resistance to environmental degradation, which lessens the need for maintenance compared to concrete and steel (Karbhari and Zhao, 2000, Kendall, 2006). However, since FRP bridges have only been on the market during the last 20 years, there are uncertainties regarding the durability of the material even though the laboratory results are promising. Concerning fatigue properties, a study presented by Liu et al. (2008) indicates good fatigue resistance after 3 million cycles. FRP profiles are prefabricated and then mounted on site which, in combination with their light weight, enables fast erection of the bridge (Bakis et al., 2002, Kendall, 2006). The drawbacks are a low bending stiffness compared to steel and a possible risk for brittle failure (Bakis et al., 2002).

Fibre reinforced polymer is a composite material consisting of polymers reinforced by some type of fibre (Fiberline Composites, 2013a). The fibres can be made of glass, carbon, or aramid, where glass fibres are the most commonly used option (Fiberline Composites, 2013c). The polymer material can consist of polyester, epoxy, or phenol, where polyester is most frequently used because of its good all round properties (Fiberline Composites, 2013d). Structurally, the fibres are used to take compressive and tensile forces while the polymer takes the shear (Fiberline Composites, 2013a).

The FRP deck considered for design in this study is produced by the manufacturer Fiberline Composites A/S. The decking system is called FBD600 Asset Bridge Deck (hereafter denoted Asset deck) and is developed for road applications with heavy vehicle loads (Fiberline Composites, 2013b).

The Asset deck is manufactured by pultrusion, a process in which the reinforcement (glass fibres) and polymer matrix are pulled through a form where the fibres are orientated correctly, then heated and cured (Fiberline Composites, 2013e). Finally the profile can be cut to the desired length.

The FRP profiles are produced in segments suitable for transportation to the building site where they are glued together to the desired width. They are then glued to the steel girders and the entire bridge superstructure is ready to be lifted into place[1].

5.2 Selection criteria for the case study bridge

An existing Swedish bridge with conventional design was chosen as a starting point for the case study. This way, a realistic context and access to data on traffic situation, investment costs etc. were obtained. In order to get as up to date data as possible, a bridge as new as possible was preferred. For the FRP bridge concept to be applicable, it was reasonable to limit the span length to maximum 30 meters in order to keep the deflection of the steel girders supporting the deck at an acceptable level. Because of the high strength/weight ratio of FRP decks, there could be a possibility to reduce the number of supports compared to the conventional bridge design. It was also necessary to exclude railway bridges since the FRP deck has problems resisting the nosing force from railway traffic[2]. Due to the quick mounting and low need for maintenance of the FRP deck, traffic disruptions causing costs and emissions can be reduced, especially on bridges with high average daily traffic (ADT) located in an urban environment. Because of the low bending stiffness of FRP decks, it was preferable to limit the width of the bridge to 10 meters. The location and conditions for this bridge will also apply to the design of the FRP bridges. A set of criteria for the bridge selection was developed based on the arguments above and these are listed below.

- Relatively new (i.e. produced in the 90's or later)
- Maximum span length of 30 m
- Road bridge
- High ADT on and/or under the bridge
- Preferably a bridge width of approximately 10 m or less
- Advantageous if the number of supports can be reduced

Based on the selection criteria, a bridge located at Ullevimotet in Göteborg was chosen.

5.3 Description of the case study bridge

The bridge at Ullevimotet, see Figure 5.1, is a flyover bridge to get on and off the European highway E6/E20 which is the main route through Göteborg. E6/E20 has an average daily traffic of almost 90,000 vehicles (Nationell Vägdatabas, 2012). The bridge leads to the city centre and carries an additional 20,000 vehicles per day. Some general data of the bridge is stated in Table 5.1. The exact location of the bridge is shown in Appendix A.

[1] Frank van der Vaart, Municipality of Utrecht, 2013-05-24
[2] Mohammad Al-Emrani, 2013-05-30

Figure 5.1. The bridge seen from E6/E20, facing north (Trafikverket, 2013).

Table 5.1. General data of the chosen bridge.

Bridge at Ullevimotet		
Service life	years	100
Bridge length	m	2x22
Bridge width	m	20
ADT on bridge	veh/day	19 715
Percentage of trucks on bridge	%	5,1%
Allowed speed on bridge	km/h	50
ADT under bridge	veh/day	87 120
Percentage of trucks under bridge	%	8,2%
Allowed speed under bridge	km/h	70

The bridge at Ullevimotet was built in 1995 and is a continuous beam bridge with steel girders in composite action with a cast in-situ concrete bridge deck (Trafikverket, 2013). It has two equally long spans of 22 meters each and is 20 meters wide. The mid support consists of four concrete columns on which the four steel girders rest, a section taken at the mid supports is shown in Figure 5.2. To ensure stability, cross beams are placed every 7.3 meters. A traditional overlay composed of polymer modified mastic asphalt (denoted PMMA) and concrete asphalt is used as surfacing. Further drawings are found in Appendix B.

The layout of the traffic on the bridge consists of two lanes towards the city centre and two lanes going onto the highway. There is also a lane for pedestrians and bicycles, as shown in Figure 5.2 and in Appendix A. Under the bridge, the highway has three lanes in each direction.

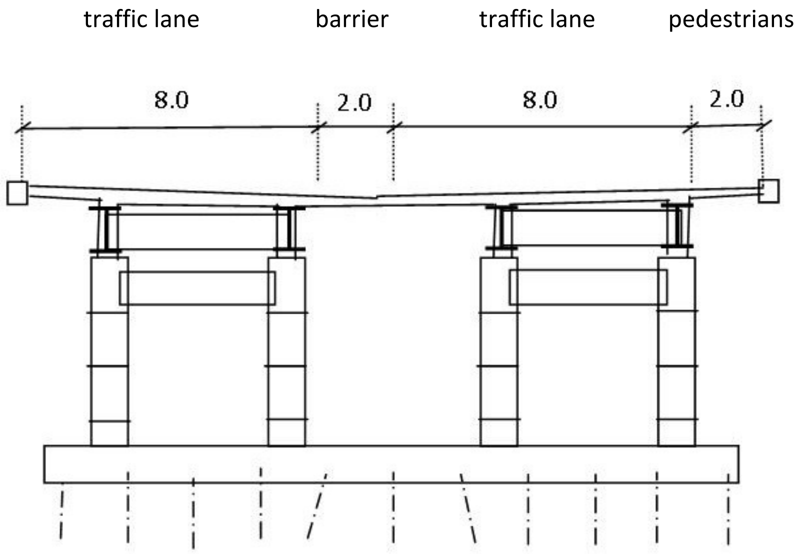
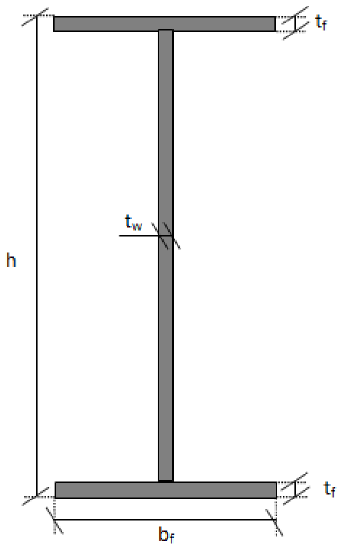


Figure 5.2. Section of the bridge at the mid support.

The concrete bridge deck has an average depth of 260 mm. The dimensions of the four steel girders vary slightly but an average girder is considered in the analysis, see Figure 5.3.



h	1080 mm
t_w	20 mm
b_{f_upper}	700 mm
b_{f_lower}	800 mm
t_f	40 mm

Figure 5.3. Average dimensions of the steel girders at the Ullevi bridge.

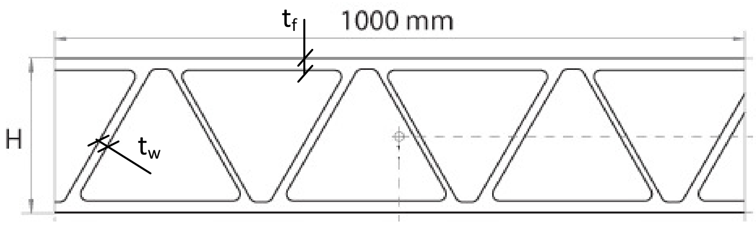
5.4 Design of bridge alternatives with fibre reinforced polymer deck

The original case study bridge is to be compared in terms of life-cycle costs and environmental impact with bridge alternatives with FRP deck. Therefore, three design options for a bridge with FRP deck have been developed. The FRP deck is resting on steel girders which are continuous over the support. Some other general aspects common for the three designs are described in the Chapter 5.4.1, 5.4.2, and 5.4.6. This is followed by individual descriptions of each bridge design in Chapter 5.4.3, 5.4.4, and 5.4.5.

5.4.1 Material properties

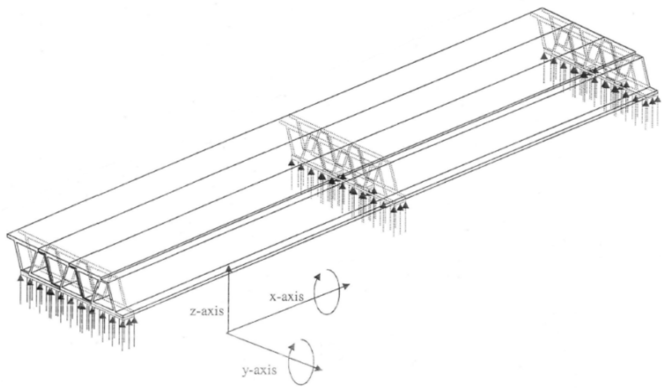
FRP DECK

The properties of the Asset deck are presented in Figure 5.4 and Figure 5.5. Usually, a FRP road bridge deck, simply supported on the girders, can span approximately three meters in its strong direction with regard to deflection, due to the relatively low bending stiffness (Liu et al., 2008, Hoffard and Malvar, 2005).



H [mm]	225
t_f [mm]	15.6
t_w [mm]	8

Figure 5.4. Section of FBD600 Asset Bridge Deck (ref. Fiberline)



E_{xx} [GPa]*	20
E_{yy} [GPa]	18
I_{xx} [mm^4/m]	342,600,000
I_{yy} [mm^4/m]	409,800,000
Weight [kg/m^2]	103.69

*E_{xx} is the modulus of elasticity in the x-direction while I_{xx} is the moment of inertia around the x-axis.

Figure 5.5. Profile of the FBD600 Asset Bridge Deck with coordinate system and sectional properties (ref. Fiberline).

Polymer concrete is the used as overlay on the FRP decks due to its light-weight and good adhesive properties to the deck. Moreover, the polymer concrete distributes the concentrated wheel loads, thus decreasing local bending of the deck (Gabler and Knippers, 2013).

STEEL

For the steel girders, steel quality S355 is used in all concepts, including the original bridge at Ullevimotet. The modulus of elasticity is 210 GPa and the density is assumed to be 7,800 kg/m^3.

5.4.2 Loading and design limits

The design of the steel girders is made considering the self-weight of the FRP-deck and the overlay as well as the traffic load. The traffic load is calculated according to load model 1 for traffic loads, LM1, in EN 1991-2, Eurocode 1.

The structures are checked regarding ultimate limit state and serviceability limit state. In the serviceability limit state, the deflection limit for the steel girders is set to L/400, as stated in Eurocode. There is no defined limit for local deflection of the deck in Eurocode, but a limit of L/300 for the FRP deck is considered as a benchmark. Load model 2, LM2, can be applied for shorter members; however this is beyond the scope of this thesis. Fatigue limit state and instability phenomena are omitted in the designs.

5.4.3 Design of alternative 1 – Transversal fibre reinforced polymer deck on steel girders

In the first design, the FRP deck is orientated transversally to the direction of the bridge, as shown in Figure 5.14. A simplified model of the FRP deck, acting as a continuous beam over the steel girders, was studied and showed that seven steel girders are needed in order to fulfil the deflection limit of the deck. The girders were therefore placed with a distance of 2.8 m as shown in Figure 5.6. Cross beams were assumed to be placed as in the original bridge at Ullevimotet. A design with an overhang on each side of the bridge of 1.2 meters and 2 meters respectively was chosen. The smaller overhang is on the side with road traffic and wider on the side with a pedestrian area, in order to match the loads. A plan sketch of the design, with traffic lanes and designing traffic load is presented in Appendix D.

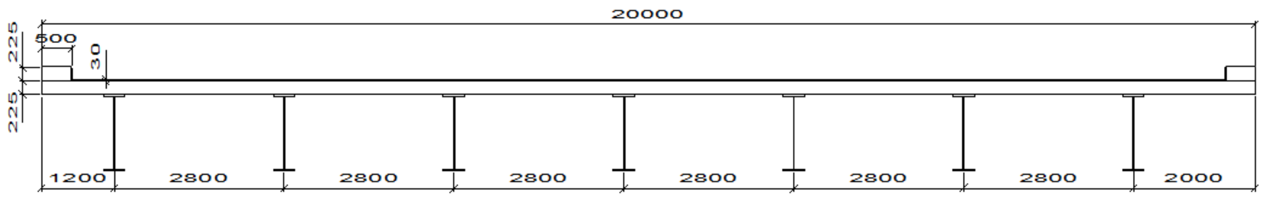
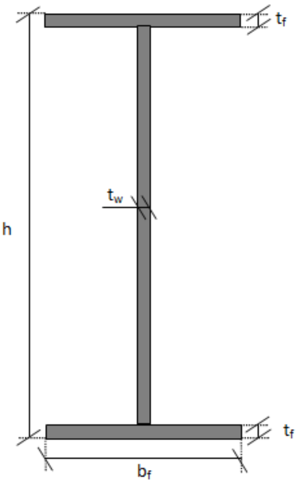


Figure 5.6. Cross section of alternative 1.

Calculations, found in Appendix D, show that the deflection of the steel girder is governing the design. The resulting dimensions of the steel girders are shown in Figure 5.7. Utilization rates for moment and shear force, weight of steel structure and exposed steel area are shown in Table 5.3.



h	1200 mm
$\mathbf{t_w}$	19 mm
$\mathbf{b_f}$	350 mm
$\mathbf{t_f}$	35 mm

Figure 5.7. Dimensions of I-girder

5.4.4 Design of alternative 2 – Longitudinal fibre reinforced polymer deck on steel girders and load-bearing cross beams

The second design alternative includes load-bearing transversal beams connected to the longitudinal steel girders. In this case, the FRP deck is orientated in the direction of the traffic flow. The purpose of using transversal beams as load-bearing elements is to allow for larger spacing between the steel girders as well as creating a plate action in the FRP deck. A simplified finite element model, using Abaqus CAE 6.12-1, showed that an acceptable deflection is achieved for a simply-supported FRP deck on four edges of 4x3.67m. Since the FRP deck in this bridge application is continuous in both longitudinal and transversal direction, it was estimated that dimensions of 3.67x4.67m would give acceptable deflections. The modelling is explained further in Appendix G. According to the results, four longitudinal steel girders with spacing 4.67 meters and load-bearing cross beams every 3.67 meters are needed. A cross section and a plan sketch of the design can be seen in Figure 5.8 and Figure 5.9 respectively.

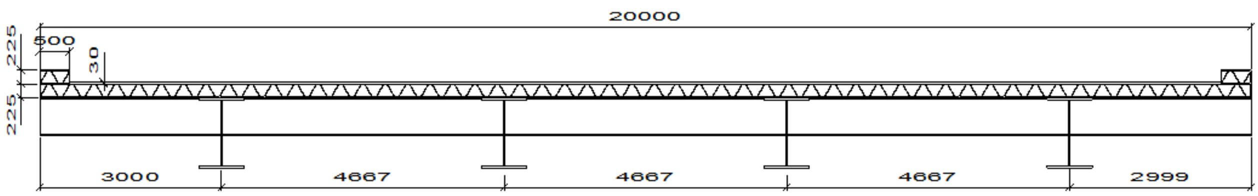


Figure 5.8. Cross section of alternative 2.

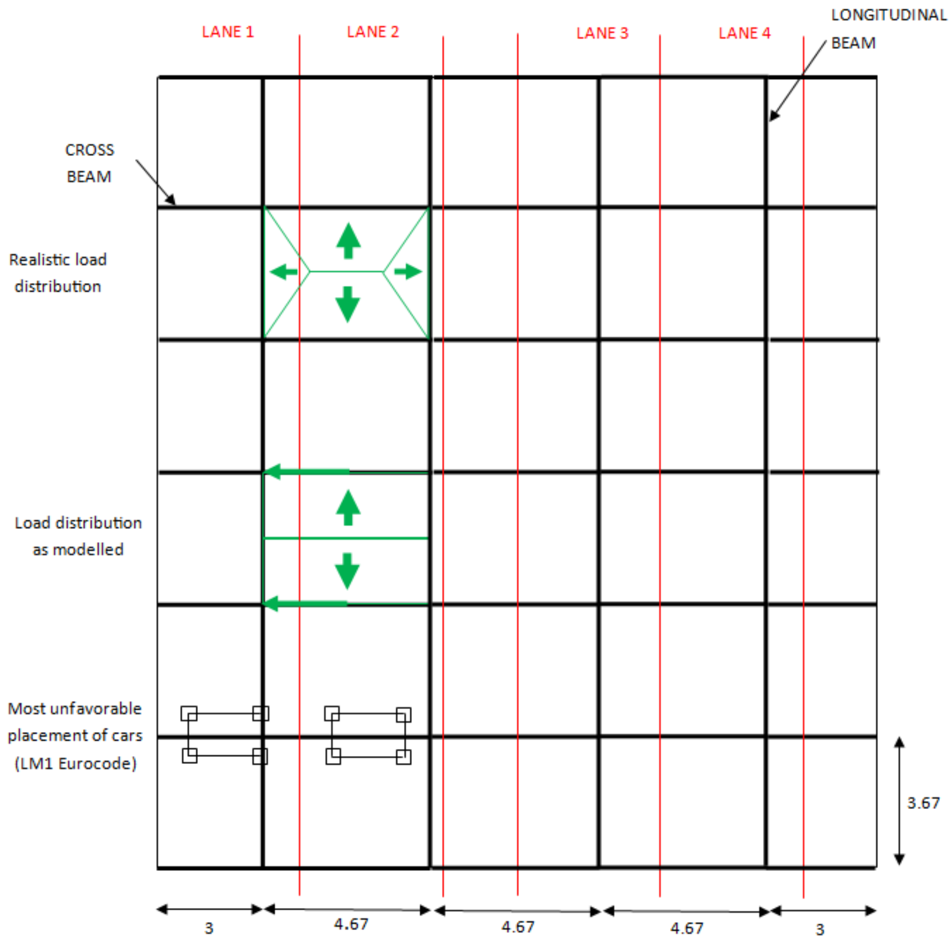
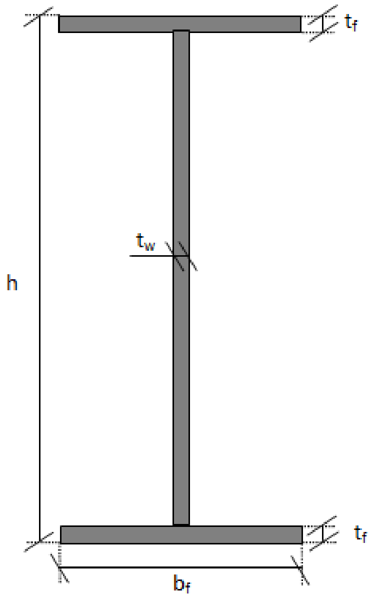


Figure 5.9. Plan sketch of alternative 2, one span. The arrows show real and modelled load distribution respectively.

In the design of cross beams and longitudinal steel girders a simplified load model was created, shown in Figure 5.9. The load model is on the safe side since all loads are assumed to act at the cross beams first, then the cross beams act as point loads on the longitudinal girders. For the cross beams, the moment capacity is governing while the deflection is the critical factor for the longitudinal steel girders. A standard profile, IPE 550, was used for the cross beams and the longitudinal girder was assigned the dimensions in Figure 5.10. A comparison of utilization rates for the longitudinal girders, weight and exposed steel area is shown in Table 5.3. Full calculations can be found in Appendix E.



h [mm]	1100
t_w [mm]	16.5
b_f [mm]	750
t_f [mm]	38

Figure 5.10. Dimensions of I-girder

5.4.5 Design of alternative 3 – Double FRP deck on steel girders

During the design of the first two alternatives with FRP deck it was observed that the required amount of steel was similar to the amount of steel in the original Ullevi bridge. In order to reduce the quantity of steel, larger spacing between the longitudinal beams was desired. Therefore, an alternative with two transversal FRP decks on top of each other was developed, thus the stiffness of the deck increased so that the spacing could be increased accordingly without compromising the deflection limit. A principal sketch of the double FRP deck is shown in Figure 5.11.

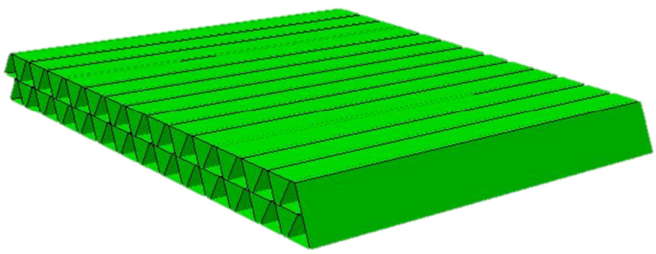


Figure 5.11. Layout of double FRP deck

The design is similar to the one of alternative 1, but an increase in the spacing between longitudinal beams from 2.8 meters to 4 meters reduced the number of steel girders from seven to five. Figure 5.12 shows a cross section of the design and a plan sketch is presented in Appendix F.

Compared to the previous FRP designs, the girder dimensions need to be increased in order to fulfil the deflection limit. There is a limit of 1.4 meters for the total height of the superstructure to keep sufficient free height between the roadway under the bridge and the bottom of the girders. This means that the height of the girders is limited to 1.0 meter to allow space for the extra FRP deck. The resulting dimensions of the steel girders are presented in Figure 5.13. A comparison to the other alternatives regarding utilization ratios, weight and exposed steel area is found in Table 5.3. Calculations can be seen in Appendix F.

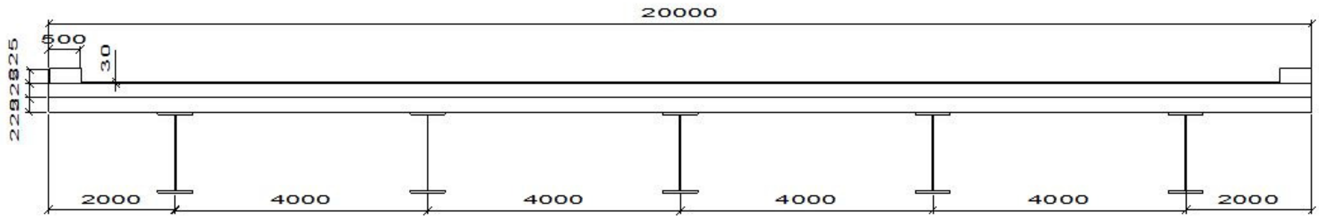
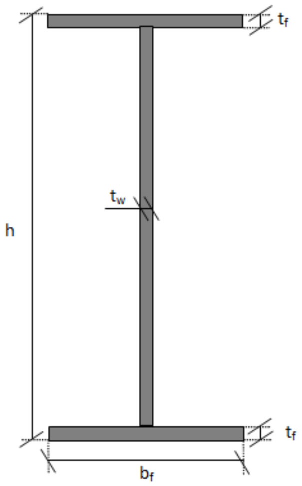


Figure 5.12. Cross section of alternative 3.



h [mm]	1000
t_w [mm]	25
b_f [mm]	850
t_f [mm]	39

Figure 5.13. Dimensions of I-girder

5.4.6 Edge beams and railings

Common for the FRP designs are the need for edge beams and railings. The solutions for the edge beams and railings were not investigated in detail since they are not critical in the design. The edge beam was designed as a thin section of the FRP Asset deck fastened on top of the main deck. An example of this solution, used at the Friedberg Bridge in Germany, is shown in Figure 5.14. If the stiffness along the edge needs to be further increased, it would be possible to attach a steel profile underneath the deck. A simplified finite element model was created in Abaqus CAE 6.12-1 to ensure that this solution was satisfying.

Bridge railings are usually fastened to concrete edge beams, which these FRP alternatives do not have. In Sweden there is also a demand for CE-certified railings. For the FRP alternatives 1 and 3 created in this case study the solution used in the Friedberg bridge in Germany is proposed, see Figure 5.15 (Gabler and Knippers, 2013). The traffic layout at the Friedberg bridge is similar to the one at the Ullevi bridge with a speed limit of 50 km/h on the bridge, and they are both overpassing a highway in an urban area. The shortcoming is that this solution might not yet be CE-certified. If a stiffer fastening would be needed it is possible to solve, for example by fastening the railings through the FRP edge beam and deck.

In alternative 2, the railings are assumed to be fastened directly to the cross beams and the rest of the design is assumed to be similar to the one at the Friedberg bridge.

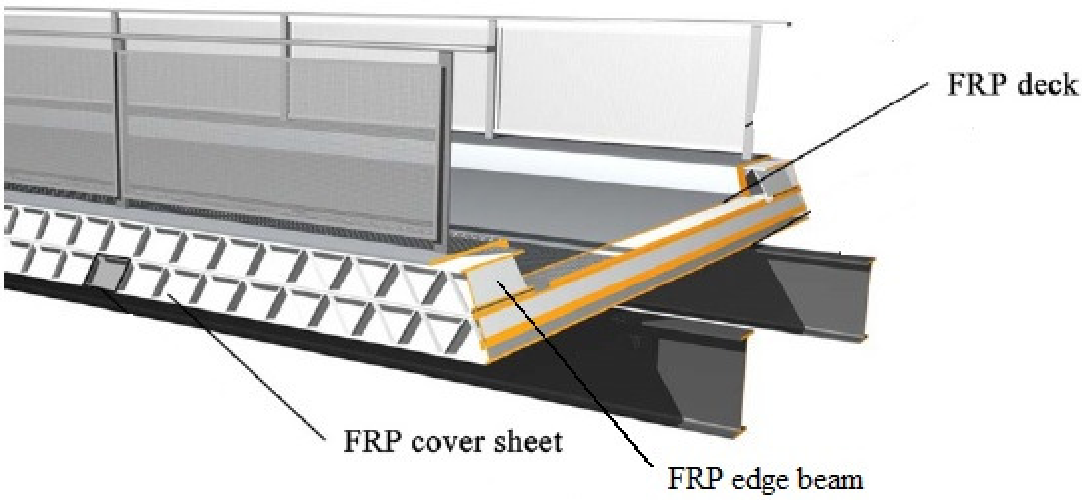


Figure 5.14. Edge beam solution used at the Friedberg bridge (Gabler and Knippers, 2013).

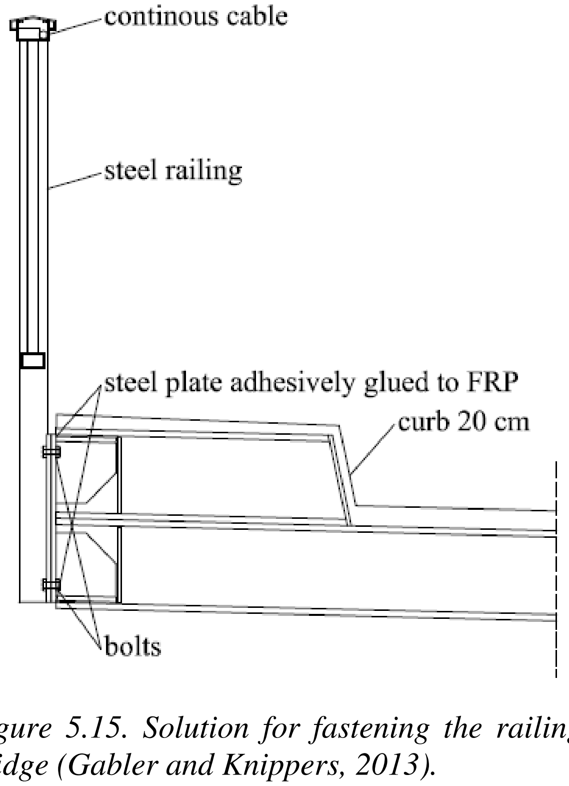


Figure 5.15. Solution for fastening the railings to the edge beam in the Friedberg bridge (Gabler and Knippers, 2013).

5.5 Design of bridge with a steel sandwich deck

A new bridge concept, which has not been built yet, is a design with a so called steel sandwich deck resting on steel girders. The steel sandwich deck has the same advantages as the FRP deck when it comes to light weight and fast assembly and is therefore suitable in urban areas where small traffic disturbances are desired. Traditionally, a sandwich deck is composed of a top and bottom plate with a core in between. However, in previous works a concept that excludes the bottom plate was developed in order to optimize the design (Alwan and Järve, 2012), and this is the concept used in this thesis.

5.5.1 Material properties

The steel quality was assumed to be S355 with a modulus of elasticity equal to 210 GPa and density 7,800 kg/m³.

All welds are performed with automatic welding; hence the weld category is at least C100 according to Table 9.8.2 in Eurocode, ENV 1993-1-1:1992.

5.5.2 Loading and design limits

The loads considered in the design were the self-weight of the deck and the surfacing and the traffic load according to load model 1 for traffic loads, LM1, in EN 1991-2, Eurocode 1.

In the ultimate limit state, moment capacity was checked but shear capacity was left out since it was assumed that it would be less critical than the moment capacity. Deflection of the steel girders was checked in the serviceability limit state and limited to L/400. Fatigue limit state was checked at the weld connecting the web and the lower flange of the girder, since this weld is subjected to the highest stress.

5.5.3 Design of primary load bearing system

Instead of having one 20 meter wide bridge, this proposal is designed as two parallel bridges with 10 meters width to facilitate the assembly on site. Each bridge carries two lanes of traffic and pedestrian areas. The proposal was modelled in Abaqus CAE 6.12-1 to make sure the load bearing capacity was sufficient. Only one span of the bridge was modelled and the span was assumed to be simply supported.

The steel sandwich deck is composed of a corrugated profile with a steel plate on top. The deck is welded to two steel girders on which the top of the web is cut to match the corrugated profile. A sketch of the design is shown in Figure 5.16 and dimensions of the corrugated profile are presented in Figure 5.17.

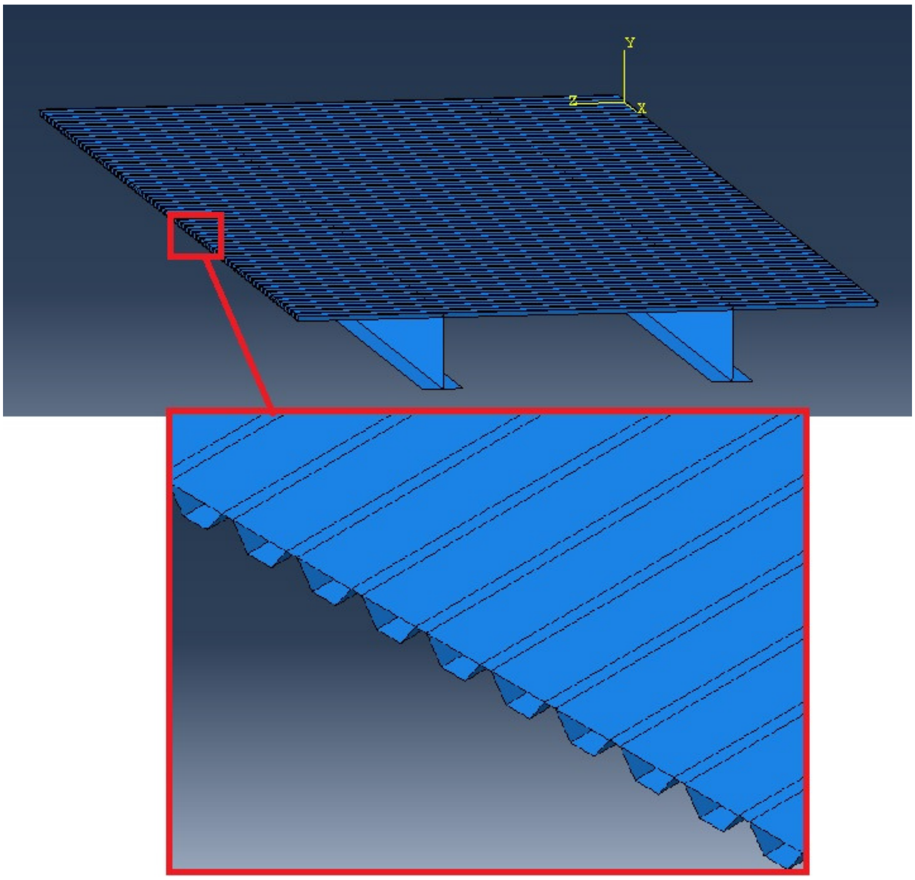


Figure 5.16. Sketch of bridge with steel sandwich deck.

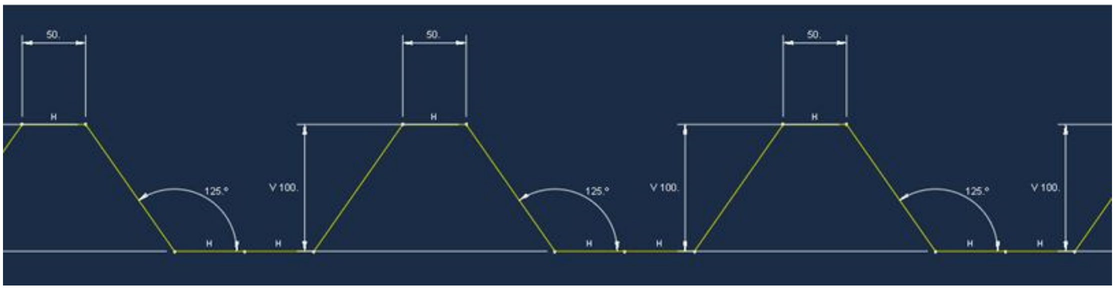


Figure 5.17. Dimensions of the corrugated profile.

In order to attach the corrugated profile to the top plate, laser hybrid welding is used. This type of weld is a combination of laser welding and gas metal arc welding (GMAW) and allows narrow and deep welds (ESAB, 2013). Welding is also needed between the deck and the web, as well as between the web and the bottom flange. Here, conventional double-sided fillet welds are used, applied using robot welding. An illustration of the welds is displayed in Figure 5.18.

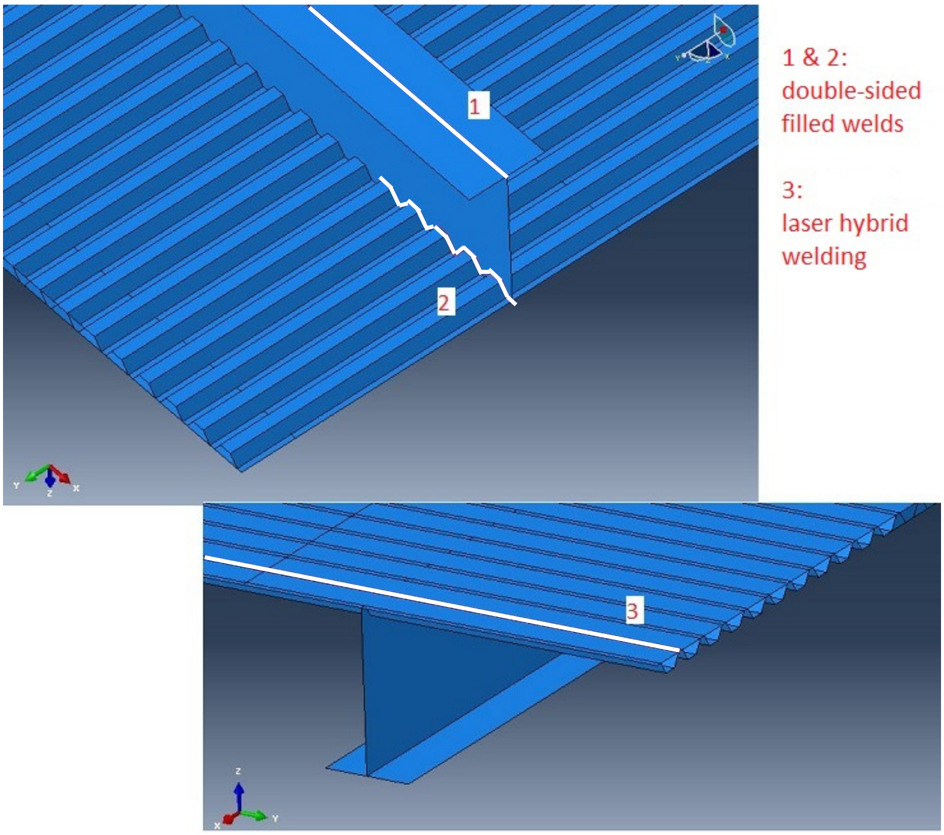
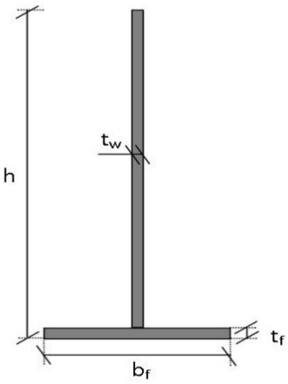


Figure 5.18. Illustration of welds (the upper picture is presented upside down to better show the welds).

The distance between the steel girders is five meters, giving an overhang of 2.5 meters on each side. The dimensions of the main girders are shown in Figure 5.19 and utilization rates, weight etc. are presented in Table 5.3. In this design, the fatigue capacity was governing the design with a utilization ratio of 91.6 %. All calculations can be found in Appendix L.



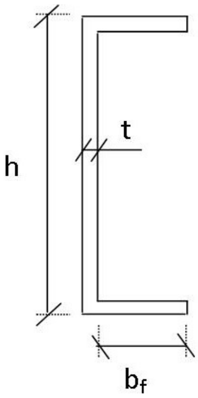
h	1300 mm
t_w	12 mm
b_f	200 mm
t_f	50 mm

Figure 5.19. Dimensions of steel girders

5.5.4 Other design details

To limit the deflection of the deck, load-bearing cross beams were placed every 7.33 meters. The cross beams between the main girders are standard profile IPE 500 while the cross beams supporting the overhand have a tapered section. A cross section of the bridge is shown in Figure 5.20. To stiffen the cantilever, a beam designed as a C-profile in steel, is placed along the edges of the bridge. Its dimensions are shown in Figure 5.21.

Figure 5.20. Cross section of the bridge.



h	600 mm
t	35 mm
b_f	130 mm

Figure 5.21. Dimensions of stiffening beam.

The surfacing of the bridge is assumed to be done as the in the recommendations for steel deck bridges in TRVR Bro 11, section G.3.3 (Trafikverket, 2011). Before applying the surfacing layer, the steel deck must be blasted in order to achieve good adhesion. The surfacing consists of five layers[3]. The first two layers are epoxy sealants, followed by a layer of insulation. On top of that, there is approximately 30 mm polymer modified mastic asphalt and a 40 mm thick layer of asphalt concrete.

Since bridges with steel sandwich decks are still on the conceptual stage, there are practical issues that need to be solved. For example, the traffic induced vibrations could cause problems both dynamically and acoustically. It might be possible to overcome such problems by filling the cavities with some sort of foam, but this has not been investigated in this case study.

5.6 Substructure

The total weights of the superstructures of the FRP and steel sandwich alternatives are substantially lower than that of the steel/concrete bridge, as can be seen in Table 5.3.

[3] Dan Aronsson, DAB AB, 2013-05-21.

Therefore, a check was made to see if the dimensions of the mid support could be reduced. The four columns of the original bridge had a diameter of 1 meter and it was found that significantly smaller dimensions were required to carry the vertical load from the superstructure of FRP alternative 1, 2, and 3 and the steel sandwich alternative. However, the governing factor in the design of the mid support was not the vertical load, but the accidental load from the traffic going under the bridge. Therefore, the columns of the FRP and steel sandwich alternatives were assumed to be equivalent to those of the steel/concrete bridge. In the alternatives with more than four longitudinal steel girders, it was assumed that lintels can be used to distribute the load to the four columns.

The existing bridge at Ullevimotet has six piles under each column at the mid support. These piles are designed for a maximum compressive force of 778 kN, giving a total load bearing capacity at the mid support of around 18.7 MN. In order to check if the number of piles could be reduced for the FRP alternatives, the load effect at the mid support was calculated for each design and compared to the original bridge. The results are presented in Table 5.2 and calculations are found in Appendix M. Since the load effect for alternative 1 and 2 was around 75% of the load effect of the steel/concrete bridge, a rough estimation was made that the number of piles could be reduced accordingly. This corresponds to a reduction with six piles, however only four piles were omitted due to symmetry. No check was made for the steel sandwich alternative but since its weight is similar to FRP alternative 3, it is assumed that the number of piles cannot be reduced.

Table 5.2. Comparison of load effects for the steel/concrete bridge and the FRP alternatives.

	Steel/concrete bridge	*FRP alternative 1*	*FRP alternative 2*	*FRP alternative 3*
Load effect [MN]	16.6	12.1	12.8	15.2
ratio to load effect of Ullevi bridge[%]	100	73	77	92

5.7 Summary of bridge designs

Table 5.3 presents a comparison of the different alternatives with regard to utilization rates, weight and exposed steel area. The exposed steel area is of interest since it affects the amount of maintenance needed.

The steel/concrete bridge at Ullevimotet was not designed according to Eurocode but using the Swedish standard applicable at the time. Therefore, a direct comparison to the developed alternatives, which were designed according to Eurocode with regard to design loads, moment and shear force etc., could not be made. However, for the loads considered in this thesis there is normally no major difference in the final dimensions depending on if the bridge is designed according to Eurocode or the old Swedish standard[4]. The weight of the steel superstructure and exposed steel area was calculated and a comparison between the alternatives is shown in Table 5.3.

Table 5.3. Comparison of all alternatives

	Ullevi bridge[1]	*FRP alternative 1*	*FRP alternative 2*	*FRP alternative 3*	*Steel sandwich*
u_{moment} [%]	N/A	69.8	68	58.9	50.7
u_{shear} [%]	N/A	16.1	29.9	23.6	-
$u_{deflection}$ [%]	N/A	98.9	98.2	98.9	62.4
m_{steel} [tonne]	118	121	126	165	260
m_{tot}[tonne]	928	272	277	408	406
A_{exp} [m²]	919	1 222	1 221	1 184	2 399

[1] Calculation of amounts used in the Ullevi bridge can be found in Appendix C

5.8 Input for life-cycle cost analysis

A life-cycle cost analysis was performed for the steel/concrete composite bridge, the FRP alternatives 1, 2 and 3 and the steel sandwich bridge. The service life for all bridge alternatives was set to 100 years and the discount rate to 3.5%. All alternatives consider the ADTs stated in Table 5.1. The input data used and the assumptions and limitations made for the LCCAs are presented in the following section. Material prices and maintenance activities are assumed to be common for all three FRP alternatives even though the quantities differ.

All LCC-calculations are made according to the methodology in Chapter 2. The calculations were computed in an Excel-sheet created by the authors, see Appendix H. The following assumptions are made:

- the costs in the planning phase were omitted
- accident costs are disregarded due to lack of reliable input data
- the vehicle operation cost and traffic delay cost are combined into one hourly cost, as in Chapter 2.2.2.
- the environmental impact, regarded as a society cost, was excluded. Instead, environmental impact is treated separately in an LCA.
- costs that are common for all alternatives, such as columns, railings and bearings were disregarded

[4] Dan Nilsson, COWI, e-mail 2013-05-14

The LCCA was divided into different phases; investment, operation and maintenance, and end-of-life. The costs in each phase can be divided according to Figure 2.2. In the investment, and operation and maintenance phases both agency costs, including material and worker costs, and user costs due to traffic disruption, are considered. The end-of-life phase only considered agency costs.

5.8.1 Agency costs

Investment phase

The prices used for the agency costs are presented in Table 5.4. The price of the FRP Asset deck, according to Sørensen[5], consists of the cost for the profiles (5292 SEK/m^2) and the cost for gluing them together (1470 SEK/m^2). Depending on the size of the order, the manufacturer Fiberline gives a discount, which in this case was 10%. The cost for preparing the surface for adhesion to the polymer concrete (252 SEK/m^2) was also added to the price for the FRP Asset deck. The cost of the polymer concrete consists of a material cost (630 SEK/m²) and a cost for the application of the surfacing (378 SEK/m^2). There is a difference in the cost of asphalt between the steel/concrete bridge and the steel sandwich concept, depending on differences in which layers are included in the price.

Table 5.4. Agency costs used as input for the LCCA, including material and working costs.

	Unit price	
Formwork[1]	550	SEK/m^2
Concrete[1]	1 800	SEK/m^3
FRP Asset deck[2]	6 338	SEK/m^2
Steel[1]	24 500	SEK/tonne
Beam IPE 600[3]	13 700	SEK/tonne
Reinforcement[1]	13 200	SEK/tonne
Insulation[4]	1 160	SEK/m^2
Asphalt concrete and PMMA, steel/concrete concept[4]	450	SEK/m^2
Polymer concrete[2]	1 008	SEK/m^2
Welding[5]	25	SEK/m
Blasting, primer, insulation, PMMA[5]	938	SEK/m^2
Asphalt concrete, steel sandwich concept[6]	200	SEK/m^2

[1] Staffan Lindén, COWI, e-mail 2013-03-04

[2] Morten Gantriis Sørensen, Fiberline Composites, e-mail 2013-04-10

[3] (Tibnor AB, 2013)

[4] (Trafikverket, 2012a)

[5] Lars-Erik Stridh, ESAB, e-mail 2013-05-12

[6] Dan Aronsson, DAB, 2013-05-21

[5] Morten Gantriis Sørensen, e-mail 2013-04-10

Operation and maintenance phase

When an LCCA is performed to find the total cost of a bridge, all activities in the operation and maintenance phase should be considered. For a steel-concrete composite bridge these activities usually are:

- Inspections*
- Cleaning of steel, edge beams and columns*/**
- Repair of drainage systems*
- Partial repair of concrete columns and edge beams*/**
- Impregnation of edge beams and columns**
- Replacement of edge beams
- Replacement of insulation
- Replacement and maintenance of bearings*
- Cleaning and replacement of expansion joints*
- Replacement of surface (asphalt) layer
- Repair of surface (asphalt) layer*
- Repainting of steel structure
- Patch painting of steel
- Painting of railings*

This case study does not aim to find the total LCC, but the difference between the proposed alternatives. Therefore, the activities which are common for the steel-concrete composite bridge and the other alternatives and are left out (marked with *). Other activities, indicated with (**), are assumed to be negligible due to low costs and/or small traffic disruptions.

The maintenance activities that were accounted for in the LCCAs are presented in Table 5.5. The intervals between maintenance activities were determined on the basis of interviews[6] whereas the costs, including both materials and workers, were retrieved from a BaTMan price list for maintenance activities (Trafikverket, 2012a) or from the interviews. Cracking is critical for the service life of polymer concrete on FRP decks (Wattanadechachan et al., 2006, Berman and Brown, 2010) and with regard to this an interval of 20 years was assumed for the replacement of overlay on FRP alternatives 1,2, and 3 (Anderson et al., 2013). Patch painting was considered for the steel sandwich bridge due to its large exposed steel area, but neglected for the other alternatives.

[6] John-Erik Fredriksson, COWI 2013-03-27, Daniel Rönnebjerg, COWI 2013-04-02, Jan-Olof Schröder, Göteborgs byggledning 2013-04-04, Tomas Svensson, COWI 2013-03-26 and Per Thunstedt, Trafikverket 2013-04-04. Minutes from the interviews are found in Appendix I.

Table 5.5. Maintenance activities for all alternatives.

Bridge	Part	Activity	Interval [yr]	Cost/unit [SEK/unit]
Steel/ concrete	Edge beams	Replacement [m]	45[1]	10 800[2]
	Overlay	Replacement of insulation [m^2]	40[1]	1 500[2]
		Replacement of asphalt [m^2]	10[1]	600[2]
	Steel	Repainting [m^2]	30[1]	1 700[2]
FRP 1,2 and 3	Overlay	Replacement of polymer concrete [m^2]	20[3]	1008[4]
	Steel	Repainting [m^2]	30[1]	1 700[2]
Steel sandwich	Overlay	Replacement of insulation [m^2]	40[1]	938[5]
		Replacement of asphalt [m^2]	10[1]	600[2]
	Steel	Repainting	30[1]	1700[2]
		Patch painting	15[1]	1500[2]

[1] see Appendix I

[2](Trafikverket, 2012a)

[3] (Anderson et al., 2013)

[4] Morten Gantriis Sørensen, Fiberline Composites, e-mail 2013-04-10

[5] Dan Aronsson, DAB, 2013-05-21

End-of-life phase

The end-of-life phase is only considered with regard to the cost of disposing the construction materials. Concrete is assumed to be crushed and used as landfill, steel is assumed to be recycled, and asphalt is assumed to be crushed and disposed. The FRP Asset deck and the polymer concrete are assumed to be disposed as the concrete. The assumed fees for the disposal methods are presented in Table 5.6.

Table 5.6. Prices for disposal of construction materials.

	Unit price	
Concrete[1]	1100	SEK/tonne
Steel[1]	-500	SEK/tonne
Asphalt[2]	40	SEK/tonne
FRP Asset deck	1100	SEK/tonne
Polymer concrete	1100	SEK/tonne

[1] Ramböll, via Valbona Mara, information from bridge project at Rokån

[2] (D.A Mattson AB, 2013)

5.8.2 User costs and traffic situation

Two different scenarios are considered, one where the bridge is assumed to be a new construction (thus, no traffic on the bridge is disturbed in the investment phase) and one where an old bridge is replaced. The first scenario was the case when the actual bridge at Ullevimotet was built. In the replacement scenario, the traffic on the bridge needs to be diverted during the entire construction process. An assumed time plan for the construction of the bridge on site is shown in Figure 5.22.

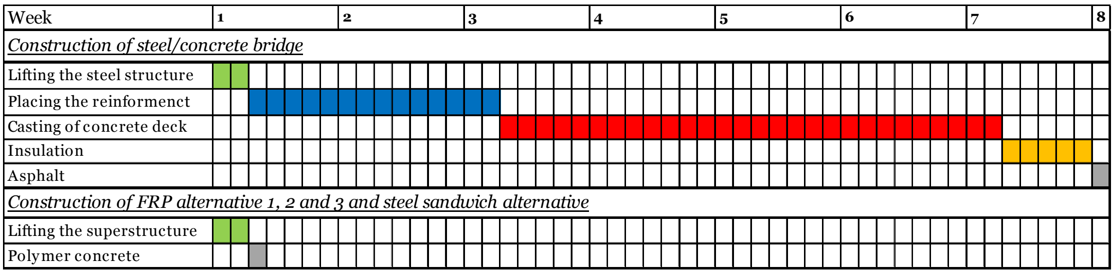


Figure 5.22. Assumed time plan for construction of the steel/concrete bridge, the FRP alternatives and the steel sandwich alternative.

The user cost included the travel delay cost and the vehicle operation cost, which combined add up to 167 SEK/h for passenger cars and 347 SEK/h for trucks. To obtain the user costs for both the investment phase and the maintenance and operation phase, assumptions regarding traffic diversions, reduced speed and queues were made. The reliability of these assumptions was verified by traffic consultant Erik Frid at COWI[7]. Generally, the speed during traffic disruptions on the bride was assumed to be reduced by 10 km/h in relation to the normally allowed speed. The time consumed during construction and for maintenance, as well as the need for closure of traffic lanes were estimated after interviews with different bridge consultants[8] and can be found in Appendix I. Traffic diversions were scheduled at night time when possible. A more detailed description of the assumptions made regarding traffic disturbances for the FRP alternatives, the steel/concrete bridge, and the steel sandwich alternative respectively is given in Table 5.7, Table 5.8, and Table 5.9. The detour routes were chosen with regard to the affected traffic volume and duration of the work and are shown in Figure 5.23.

Table 5.7. Assumptions regarding traffic disturbances during maintenance activities for the FRP alternatives.

	Activity	disruption time [h]	Traffic disruption	Detour route	length of detour [m]	reduced speed [km/h]	normal route [m]
FRP	Reparations of edge beams	2 nights	1 closed lane at a time	no detour	200 m work zone	30	200
Overlay	Replacement of surfacing, north lane	1 night	1 closed direction on bridge	Korsvägen, 50% of traffic	1 609	40	1 695
				Friggagatan, 25 % of traffic	2 977	55	1 695
				Mårten Krakowgatan, 25% of traffic	4 153	65	2 837
	Replacement of surfacing, south lane	1 night	1 closed direction on bridge	Fabriksgatan	1 255	40	511
Steel	Repainting	2 nights	1 closed lane at a time under bridge	no detour	200 m work zone	50	200

[7]Erik Frid, COWI 2013-04-11

[8] John-Erik Fredriksson, COWI 2013-03-27, Daniel Rönnebjerg, COWI 2013-04-02, Jan-Olof Schröder, Göteborgs byggledning 2013-04-04, Tomas Svensson, COWI 2013-03-26 and Per Thunstedt, Trafikverket 2013-04-04. Minutes from the interviews are found in Appendix I.

Table 5.8. Assumptions regarding traffic disturbances during maintenance activities for the steel/concrete bridge.

	Activity	disruption time [h]	Traffic disruption	Detour route	length of detour [m]	reduced speed [km/h]	normal route [m]
Edge beam	Replacement, north lane	6.5 weeks	1 lane closed on bridge	no detour	200 m work zone	30	200
	Replacement, south lane	6.5 weeks	1 lane closed on bridge	no detour	200 m work zone	30	200
Overlay	Replacement of insulation, north lane	1.5 weeks	1 closed direction on bridge	Korsvägen, 50% of traffic	1 609	40	1 695
				Friggagatan, 25 % of traffic	2 977	55	1 695
				Mårten Krakowgatan, 25% of traffic	4 153	65	2 837
	Replacement of insulation, south lane	1.5 weeks	1 closed direction on bridge	Fabriksgatan	1 255	40	511
	Replacement of surfacing, north lane	1 night	1 closed direction on bridge	Korsvägen, 50% of traffic	1 609	40	1 695
				Friggagatan, 25 % of traffic	2 977	55	1 695
				Mårten Krakowgatan, 25% of traffic	4 153	65	2 837
	Replacement of surfacing, south lane	1 night	1 closed direction on bridge	Fabriksgatan	1 255	40	511
Steel	Repainting	2 nights	1 closed lane at a time under bridge	no detour	200 m work zone	50	200

Table 5.9. Assumptions regarding traffic disturbances during maintenance activities for the steel sandwich alternative.

	Activity	disruption time [h]	Traffic disruption	Detour route	length of detour [m]	reduced speed [km/h]	normal route [m]
Overlay	Replacement of insulation, north lane	1.5 weeks	1 closed direction on bridge	Korsvägen, 50% of traffic	1 609	40	1 695
				Friggagatan, 25 % of traffic	2 977	55	1 695
				Mårten Krakowgatan, 25% of traffic	4 153	65	2 837
	Replacement of insulation, south lane	1.5 weeks	1 closed direction on bridge	Fabriksgatan	1 255	40	511
	Replacement of surfacing, north lane	1 night	1 closed direction on bridge	Korsvägen, 50% of traffic	1 609	40	1 695
				Friggagatan, 25 % of traffic	2 977	55	1 695
				Mårten Krakowgatan, 25% of traffic	4 153	65	2 837
	Replacement of surfacing, south lane	1 night	1 closed direction on bridge	Fabriksgatan	1 255	40	511
Steel	Repainting	2 nights	1 closed lane at a time under bridge	no detour	200 m work zone	50	200
	Patch painting	assumed to be done at night, flexible --> no traffic disruptions					

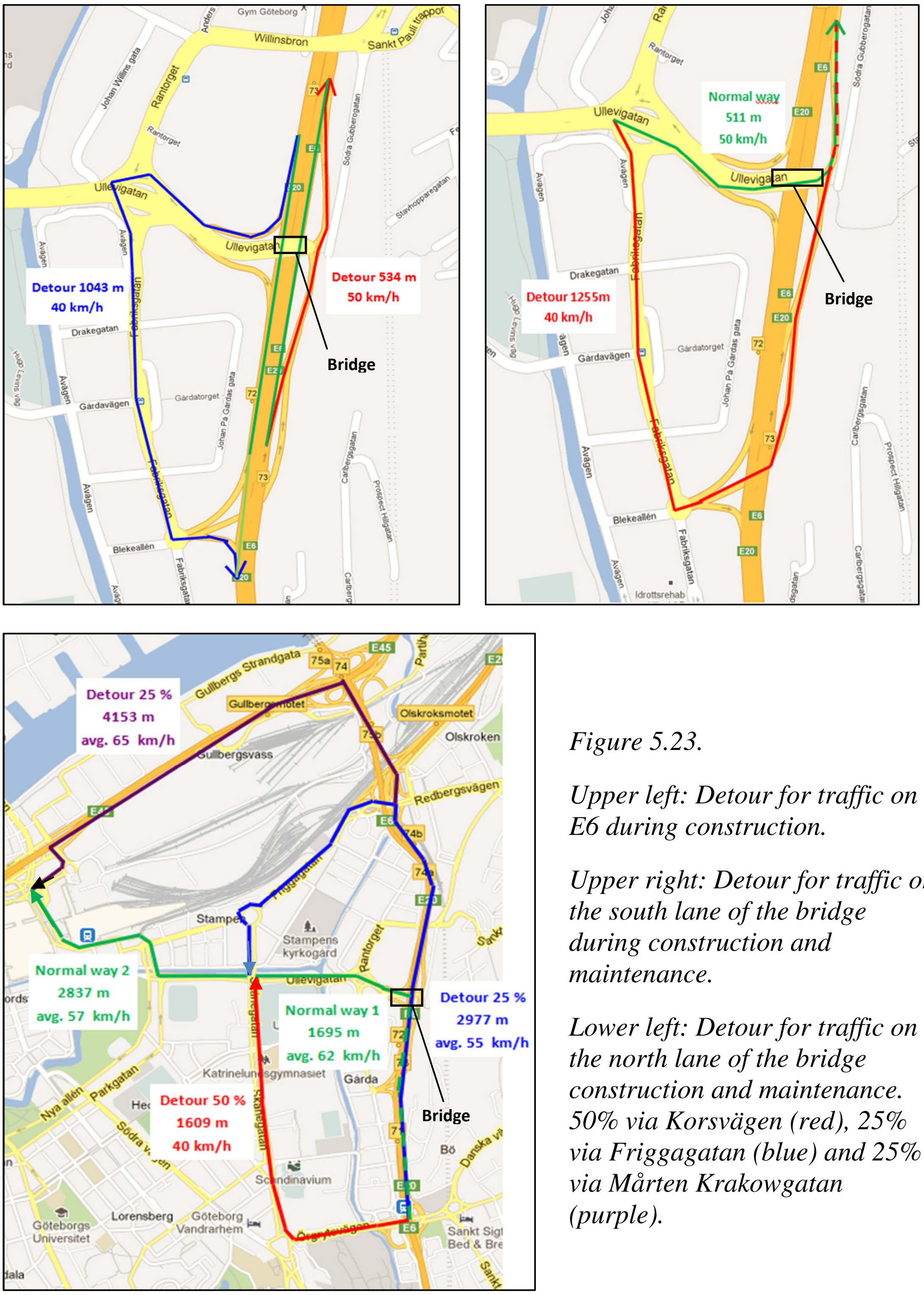


Figure 5.23.

Upper left: Detour for traffic on E6 during construction.

Upper right: Detour for traffic on the south lane of the bridge during construction and maintenance.

Lower left: Detour for traffic on the north lane of the bridge construction and maintenance. 50% via Korsvägen (red), 25% via Friggagatan (blue) and 25% via Mårten Krakowgatan (purple).

5.9 Sensitivity analysis

A life-cycle cost analysis includes a number of assumptions and uncertainties. It is important to consider these uncertainties using for example Monte Carlo simulations or a sensitivity analysis, as described in Chapter 2.3.3. In this case study sensitivity analyses were performed to investigate the following parameters, due to either an expected high impact on the total cost or uncertain assumptions:

- Discount rate
- Price of FRP Asset deck
- Traffic volume
- Interval for replacement of edge beams
- Replacement of FRP deck
- Reduced speed due to traffic disturbances
- Hourly costs for passenger cars and trucks
- Replacement of concrete deck

Discount rate: As stated in Chapter 2.2, the discount rate recommended by Trafikverket is 3.5%. In the sensitivity analysis, the discount rate is varied between 0% and 8%, partly since Trafikverket's recommendations have changed during recent years and partly since other countries use different recommendations.

Price of FRP: Fiberline Composite's price of 6,338 SEK/m^2 was used for the FRP deck. In the sensitivity analysis the price is varied between 3,000 SEK/m^2 and 8,000 SEK/m^2. This span is chosen since a decrease in price is probable if FRP usage is spread so that larger quantities are produced and the production process is further developed. The increased price is considered to account for a possible increase of raw material prices.

Traffic volume: The traffic volume on the bridge is 19,715 vehicles per day. A change in traffic volume is included to see how the LCC is affected by a change in the traffic situation at Ullevimotet. Moreover, it was of interest to see how the LCCA applies to a bridge in a different traffic context. The average daily traffic is therefore varied between 1,000 vehicles/day and 60,000 vehicles/day.

Replacement of edge beam: There are different opinions on the average service life of an edge beam. On existing bridges, the edge beams are replaced after approximately 40 years but with today's improved concrete quality, experts suggest a life span between 50 years and 80 years for the edge beam. Therefore the interval for replacement of edge beam was varied between 30 and 80 years in the sensitivity analysis.

Replacement of FRP deck: According to the FRP deck manufacturers, the service life of the deck should be around 80-100 years since the FRP deck is resistant to fatigue and deterioration. However, since FRP is a new material in bridge construction it is uncertain to predict the service life of the deck. Therefore, a replacement of the deck after 35, 50, 65, and 80 years was included in the sensitivity analysis.

Reduced speed: One of the greatest uncertainties in the LCCA is how the traffic situation is affected by construction activities. Since both detour routes and the speed on these are estimated, a sensitivity study is necessary. A speed reduction between 5 km/h and 30 km/h is investigated.

Hourly costs: In this study, the hourly cost for passenger car and truck includes both vehicle operation cost and traffic delay cost. Since there are no officially recommended values for this, there could be large variations, depending on how time is valued. The hourly cost for a passenger car is thus varied between 0% and 200% from the originally used value, where 0% does not take user costs into account at all and 200% shows how the LCC is influenced if the time value is significantly upgraded.

Replacement of concrete deck: Normally, replacement of the concrete deck in a composite bridge should not be necessary. However, the results of a survey in the PANTURA project indicate that a common problem associated with existing bridges is deterioration of concrete decks[9]. Therefore, a replacement of the deck after 50 years is included in the sensitivity analysis.

5.10 Input for life-cycle assessment

Defining a functional unit is an essential step in the life-cycle assessment. In this case study, the functional unit is a road bridge with four traffic lanes, pedestrian area and a life span of 100 years.

The life-cycle assessments included environmental impact from the construction materials and emissions caused by traffic disruptions during the entire service life of the bridge. Transportation of the materials from gate to site and from site to deposition was excluded due to limitations in the software tool used. The amounts of materials were taken from the preliminary designs of the bridge alternatives, taking into account material usage in all phases of the life-cycle. Like in the LCCA, materials which are the same for all alternatives, like railings and supports, were excluded from the analysis.

The total traffic delay caused by disruptions was the same in the LCAs and the LCCAs. However, to be compatible with the LCA software, all delays during the service life had to be converted into one total delay.

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[9] Reza Haghani, *FRP composites in construction,* Workshop FRP Bridges, Chalmers University of Technology, 2013-05-24.

To perform the LCAs, ETSI's software BridgeLCA was used. BridgeLCA is an Excel-based software and input is given in two steps. In the first step, the software considers the amounts of all used materials and the transport of those materials that are considered to have major LCA impact. Moreover, it is possible to take the amounts of electricity and diesel consumed into account.

The major limitation in this step is that it is not possible to include transports of user defined materials, in this case FRP and polymer concrete. Therefore, to achieve a fair comparison of the different alternatives transportation of materials was excluded from the analysis. In a case study carried out at the Spanish company Acciona, it was shown that the environmental impact from transport and machinery during the construction of a concrete bridge dominated the total impact in the LCA (Guedella Bustamante, 2013). Therefore, the exclusion of transports should be kept in mind when interpreting the results.

The second step in BridgeLCA comprises of input for traffic disruptions and gives output in form of carbon dioxide and sulphur dioxide. As mentioned above, all delays needed to be converted into one since it is only possible to use one detour/closure of the road in BridgeLCA while the LCC analyses considers several different scenarios. In this case study, no detour is specified in BridgeLCA, but the total delay is instead calculated using a reduced speed in a specific work zone. In principle, the reduced speed and the length of the work zone is found according to Equation 10 and then used as input for BridgeLCA. The input for the normal case, when the bridge is open, was calculated in the same way. Regarding the type of traffic, it is assumed that all heavy traffic was trucks, no buses, and that the percentage of passenger cars running on petrol and diesel respectively is as in Sweden 2012 (Trafikanalys, 2012)).

$$\Delta t_{mean} = v_{red} * s_{wz} \tag{10}$$

where

Δt_{mean} is the mean traffic delay time from the LCCAs

v_{red} is the assumed reduced speed, input for Bridge LCA

s_{wz} is the length of the work zone, input for Bridge LCA

BridgeLCA uses emission vectors from the ecoinvent database and considers eight different impact categories at midpoint level (Brattebo and Reenaas, 2012). These are global warming potential (GWP), ozone depletion (ODP), terrestrial acidification (AP), freshwater eutrophication (EP), fossil depletion (FD), human toxicity cancer (HTC), human toxicity non-cancer (HTNC) and ecotoxicity (ET). The first five are calculated according to the ReCiPe method, v 1.06, while the last three are calculated with the USEtox method. The results of the LCA are then shown category by category. Moreover, a normalization of the categories GWP, ODP, AP, EP, and FD is conducted based on the population of Europe. The toxicity categories are excluded from the normalization step since the methods for this are uncertain.

A critical part of the LCA analyses was to add the materials FRP and polymer concrete to BridgeLCA. In order to do this emission vectors are needed, including the same categories as the ones used in BridgeLCA. The emission vectors for FRP and polymer concrete are found using the software openLCA and the ecoinvent database. In openLCA, the methods ReCiPe (H) and USEtox are used respectively to obtain the emission vectors. The mix design for the FRP and for two different resins for polymer concrete is shown in Table 5.10. Out of the two resins, one is polyester based and the other is based on epoxy. Today, the epoxy based resin is used for polymer concrete overlays for Fiberline Composite's decks in Denmark[10] while polyester based resins are used as overlay on roads in the United States (Anderson et al., 2013). Both resins are considered for use in Sweden. Since a polyester based polymer concrete is used in the LCCAs, the same is used in the LCAs as well. However, the epoxy based polymer concrete emission vector is also computed in order to evaluate the difference in environmental impact compared to polyester based polymer concrete.

The optional step weighting is considered in order to investigate how weighting affects the final results. However, the weighted results will not be used as a base for decision due to the subjectivity in the weighting process, as described in Chapter 3.1.3. Four different sets of weighting factors are applied, namely Harvard, BEES default, EDIP, and US-EPA, and their factors are given in Table 3.1.

Table 5.10. Mix design for FRP, polyester polymer concrete and epoxy polymer concrete used in openLCA.

Material	Component	Proportion [% wt.]
FRP	Glass fibres	80[1]
	Unsaturated polyester resin	20[1]
Polymer concrete, polyester	Unsaturated polyester resin	20[2]
	Natural aggregates	80[2]
Polymer concrete, epoxy	Epoxy resin	13.5[3]
	Natural aggregates	86.5[3]

[1] Benedikte Jørgensen, Fiberline mail 2013-04-10

[2] (Martinez-Barrera et al., 2011)

[3] (HIM, 2001) and (HIM, 1997)

[10] Morten Gantriis Sørensen, Fiberline Composites, e-mail 2013-04-30

6 Results

A life-cycle cost analysis and a life-cycle assessment were performed for the conventional steel/concrete composite bridge, the steel sandwich bridge and the three FRP alternatives. The results are presented in Chapter 6.1 and Chapter 6.2.

6.1 Results from the life-cycle cost analyses

Two different scenarios were evaluated. In the first scenario, it was assumed that there was no bridge at the location before construction and therefore only traffic running below the construction site will be affected during the construction. In scenario number two, it was assumed that a bridge that was currently located at the site would be replaced. This demands rerouting of the traffic on the bridge while the old bridge is demolished and the new bridge is constructed, as explained in Chapter 5.8.2.

6.1.1 Scenario 1 – New bridge construction

The total costs for all bridge designs are presented in Figure 6.1. In total, the steel/concrete bridge has the lowest total cost, followed by FRP alternatives 1, 2 and the steel sandwich alternative which are very similar in price. Alternative 3 is the most expensive design due to the cost for double FRP decks. The user cost for the FRP alternatives is just above SEK 10 000 while it is a couple of hundred thousand for the steel/concrete and steel sandwich alternatives. However, the user cost is negligible for all alternatives in this scenario as can be seen in Figure 6.1.

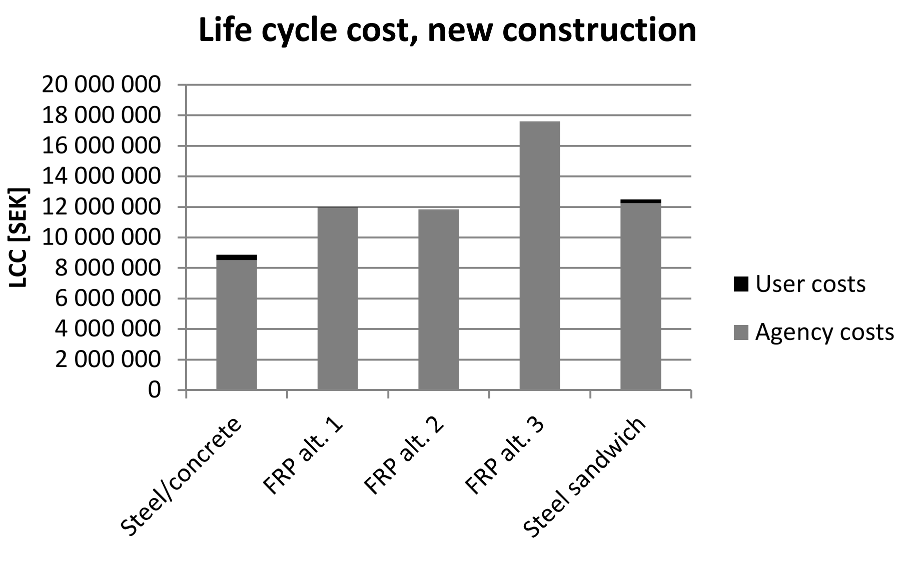

Figure 6.1. Life-cycle cost for each design, scenario 1.

For each bridge design, the distribution of costs over the life-cycle phases is presented in Figure 6.2. It is clear that the investment cost composes a larger part of the total cost for the FRP alternatives than for the steel/concrete bridge and the steel sandwich bridge. The end-of-life phase is negligible for all designs.

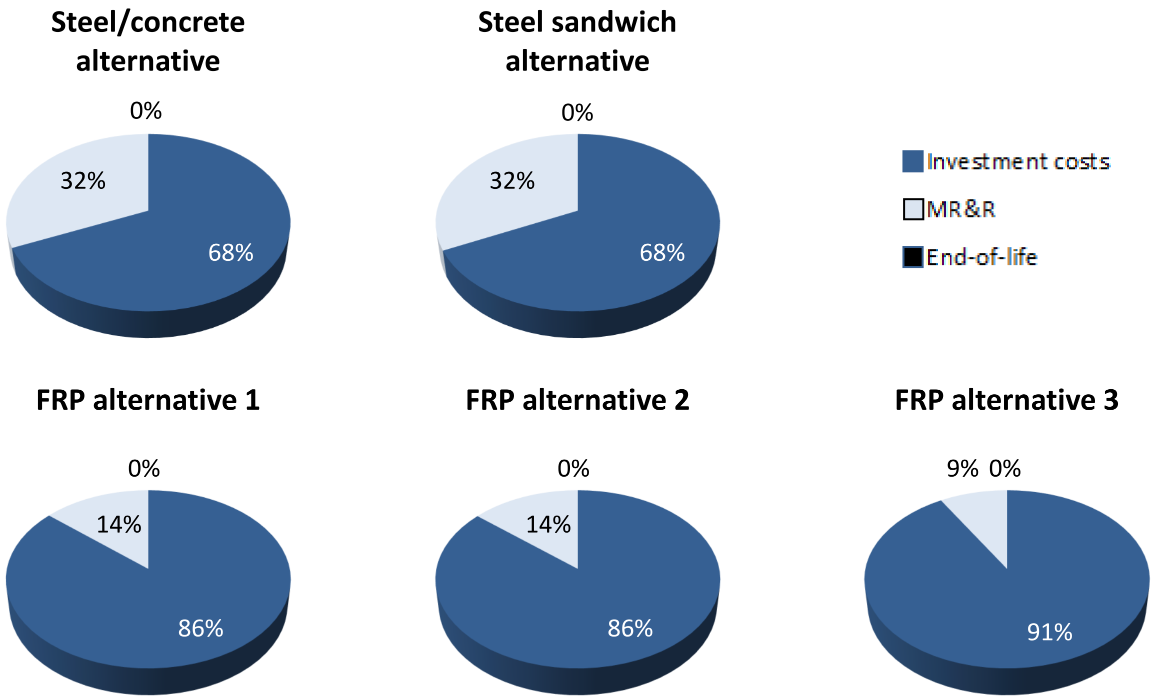

Figure 6.2. Cost distribution over the life-cycle for all bridges in scenario 1.

The cost in the operation and maintenance phase is composed of agency costs and user costs for the maintenance activities, as shown in Table 6.1 and Figure 6.3. For the FRP alternatives, the major cost is the cost for repainting and all user costs are negligible. For the steel/concrete bridge, replacement of surfacing composes the largest part of the total present cost due to the tight interval between the maintenance occasions. Replacement of insulation has a high cost each time but when converted to present value the cost decreases significantly. For the steel sandwich alternative the repainting form a large cost due to the high agency costs.

Table 6.1. Break down of costs in the operation and maintenance phase, an example is found in the figure below.

	Maintenance activity	Agency cost each time	User cost each time	Total present cost
FRP alternatives 1,2 and 3	Reparation of edge beam/deck	781	441	1 156
	Replacement of surfacing	498 960	3 283	475 054
	Repainting	2 077 400	885	1 098 242
Steel/concrete bridge	Edge beam replacement	950 400	443 641	359 500
	Replacement of insulation	1 188 000	723 369	604 691
	Replacement of surfacing	475 200	3 283	1 112 629
	Repainting	1 562 300	885	826 044
Steel sandwich	Replacement of insulation	825 000	723 369	489 850
	Replacement of surfacing	528 000	3 283	1 068 366
	Repainting	4 079 116	885	2 156 022
	Patch painting	359 922	-	318 645

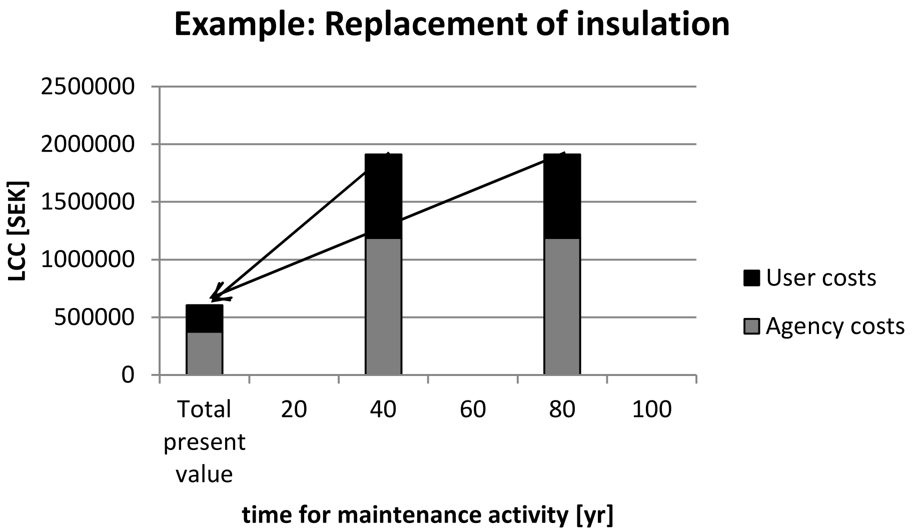

Figure 6.3 Example of how the costs are transferred to present value for replacement of insulation. All other activities can be found in the table above.

6.1.2 Scenario 2 – Replacement of bridge

The total costs for all bridge designs are presented in Figure 6.4. In total, FRP alternative 2 had the lowest cost, but the final cost is similar for all alternatives except FRP alternative 3 which, as in scenario 1, has the highest total cost. The steel/concrete bridge has the lowest agency cost but loses towards FRP alternative 1 and 2 due to high user costs.

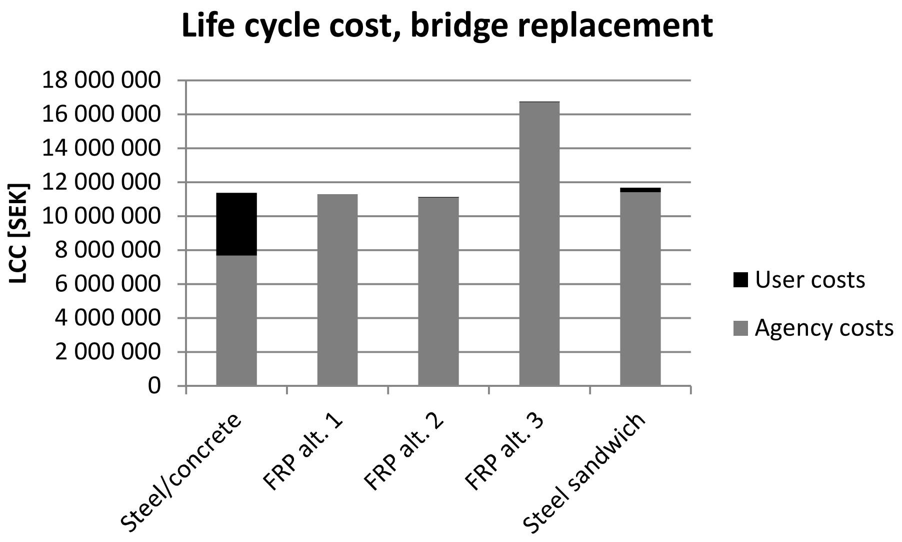

Figure 6.4. Total cost for each design, scenario 2.

For each bridge design, the distribution of costs over the life-cycle phases is presented in Figure 6.5. Just as in scenario 1, the investment cost composes a larger part of the total cost for the FRP alternatives than for the steel/concrete bridge and steel sandwich bridge. The steel sandwich bridge has the largest portion of maintenance costs. The end-of-life phase is negligible for all designs.

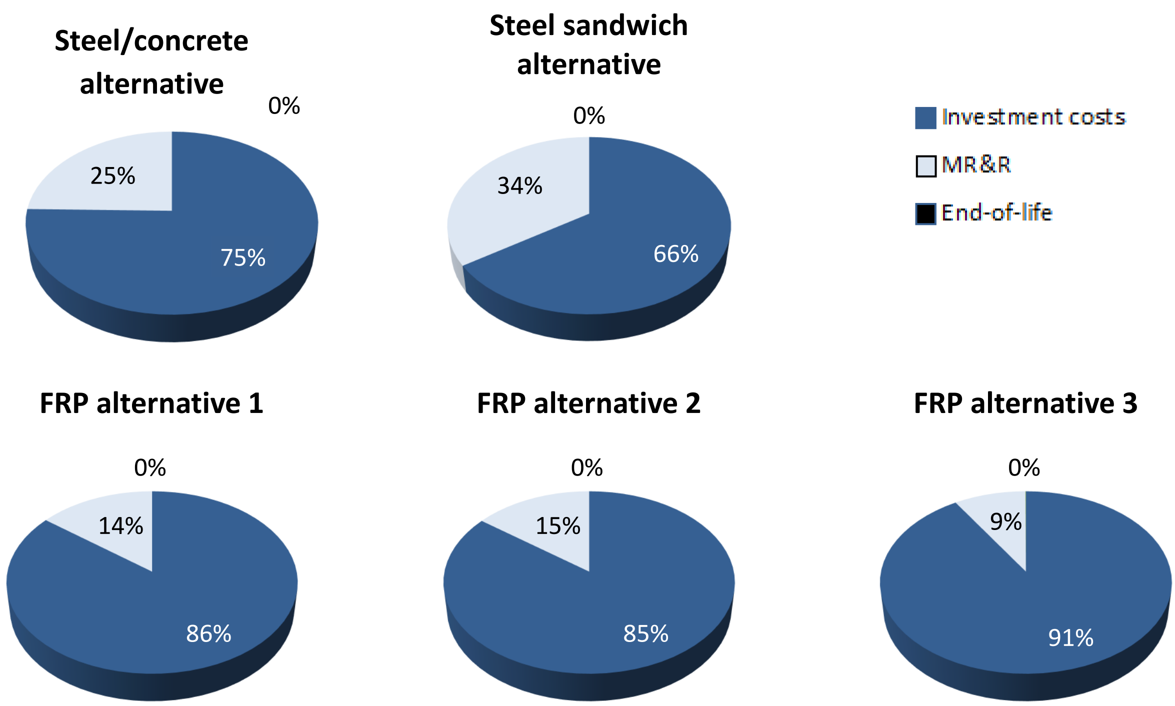

Figure 6.5. Cost distribution over the life-cycle for all bridges in scenario 2.

6.1.3 Sensitivity analysis

The sensitivity analysis was performed considering a number of different parameters, according to Chapter 5.9. The results are presented in the following sections.

Discount rate

In the first study, the influence of a varying discount rate was investigated; see Figure 6.6 and Figure 6.7. The tendency is that the FRP alternatives are more profitable the lower the discount rate is, i.e. when more consideration is taken for future costs. In scenario 1, FRP alternative 1 and 2 are cheaper than the steel/concrete bridge for discount rates around and below 1 % and for scenario 2 the breaking point is around 6 %, for discount rates above 6 % the price is more or less the same for all alternatives except FRP alternative 3. It is evident that the total cost is only sensitive to a change of discount rate when the discount rate is below about 4 %. The order of the steel/concrete and steel sandwich alternatives is unaffected by the discount rate.

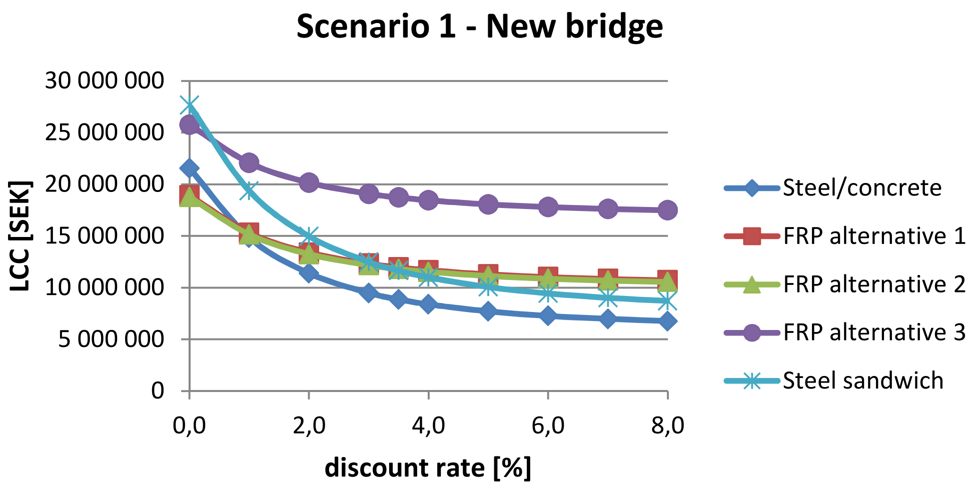

Figure 6.6. Effect of varying discount rate for scenario 1.

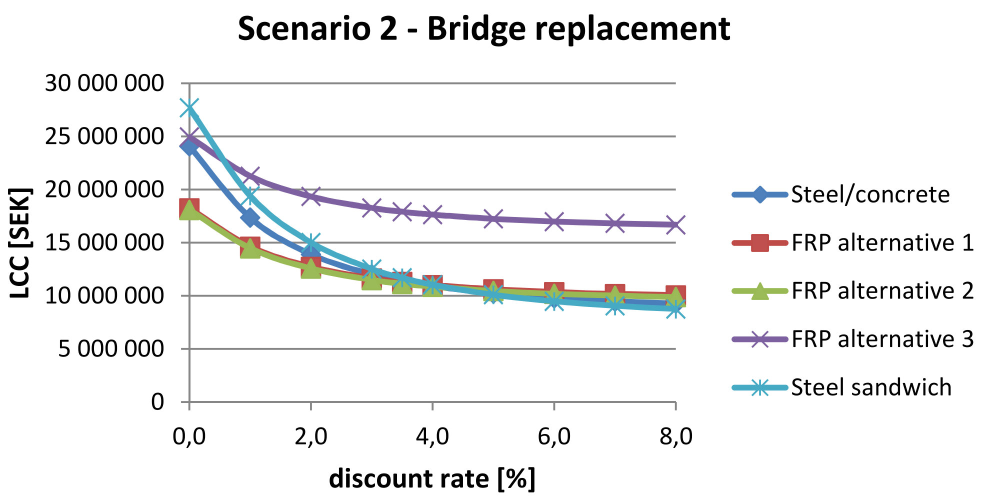

Figure 6.7. Effect of varying discount rate for scenario 2.

Price of FRP deck

The price of the FRP Asset deck was the next parameter to be analysed, the results are illustrated in Figure 6.8 and Figure 6.9. In scenario 1, the FRP alternatives are profitable for prices of approximately SEK 3,000, corresponding to a 55 % decrease of today's price. In scenario 2, the FRP price needs to increase to around SEK 6,500 for the steel/concrete bridge to be cheapest.

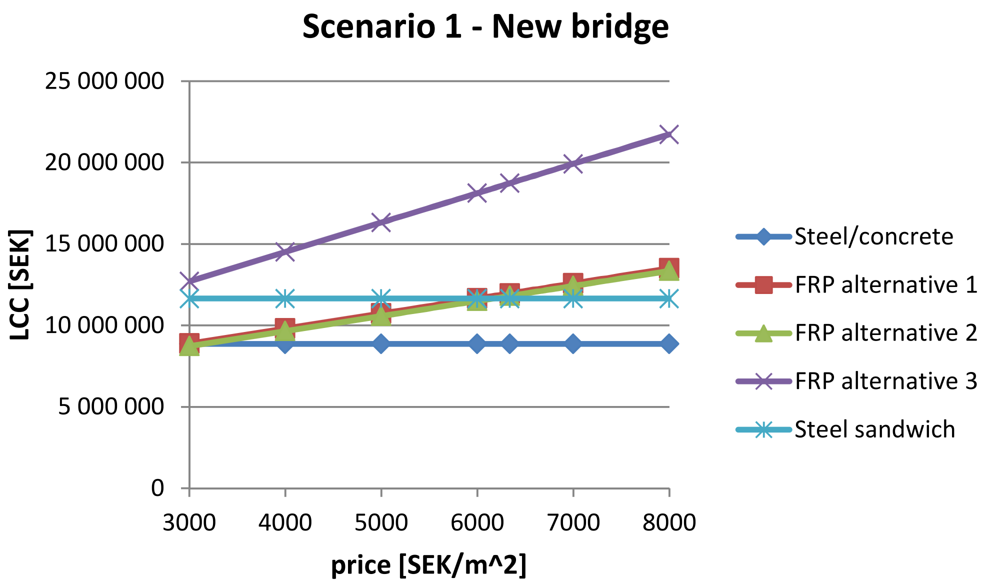

Figure 6.8. Effect of varying FRP Asset deck price, scenario 1.

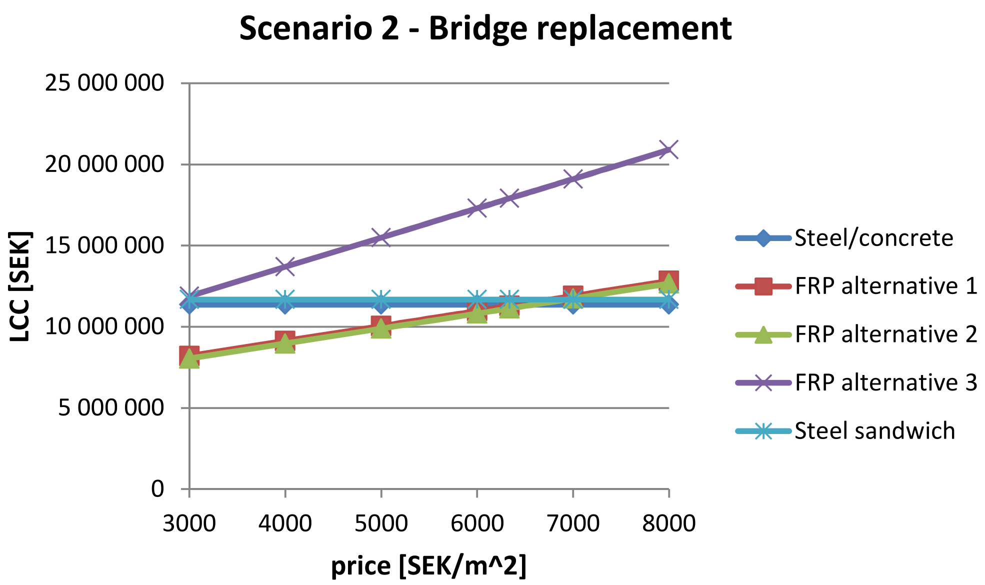

Figure 6.9. Effect of varying FRP Asset deck price, scenario 2.

Traffic volume

The third investigated parameter was the traffic volume; the results can be seen in Figure 6.10 and Figure 6.11. There is a linear ADT/cost relationship. Since the user cost for the FRP alternatives and the steel sandwich alternative only form a small portion of the total cost, their total prices are nearly unaffected by a change in traffic volume. In scenario 1 the portion of user costs is small for the steel/concrete bridge as well and its LCC is lower than the other alternatives for the whole range investigated. In the case of replacement of the bridge, scenario 2, the intersection between the LCC for the steel/concrete bridge and the FRP alternative 2 is at around 15,000 vehicles/day.

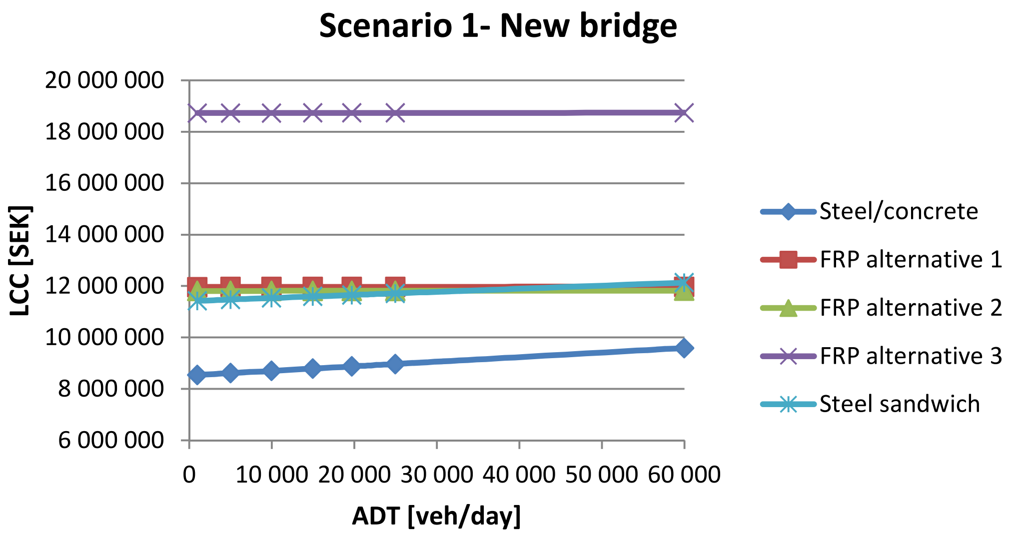

Figure 6.10. Effect of varying traffic volume, scenario 1.

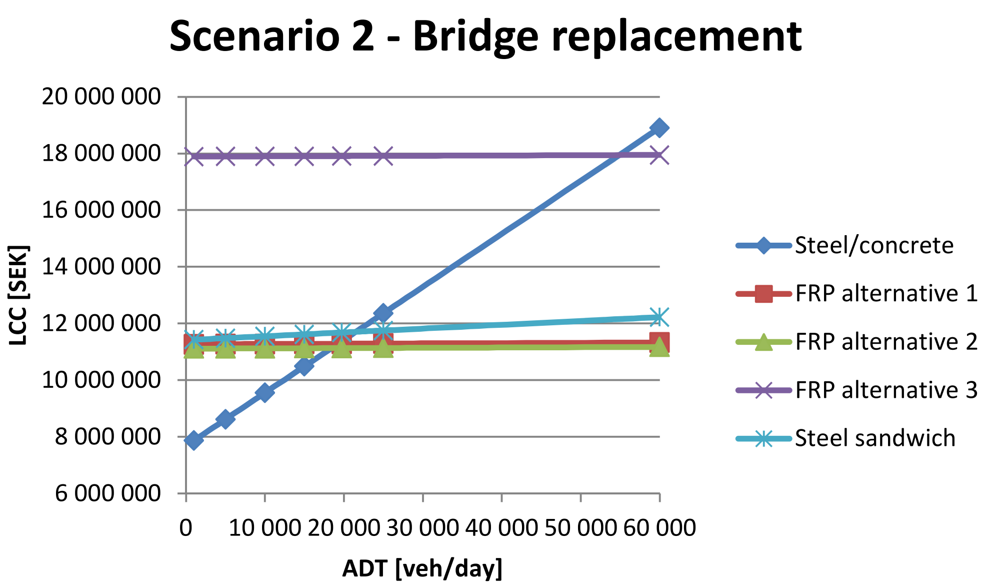

Figure 6.11. Effect of varying traffic volume, scenario 2.

Hourly cost for passenger cars and trucks

The influence of a changed hourly cost for passenger cars and trucks was evaluated, as shown in Figure 6.12 and Figure 6.13. Since the steel/concrete bridge has a larger share of user costs than the other designs, the impact of changed hourly cost is greater for this alternative. The breakeven point between FRP alternative 2 and the steel/concrete bridge, for scenario 2, is around 150 SEK/h.

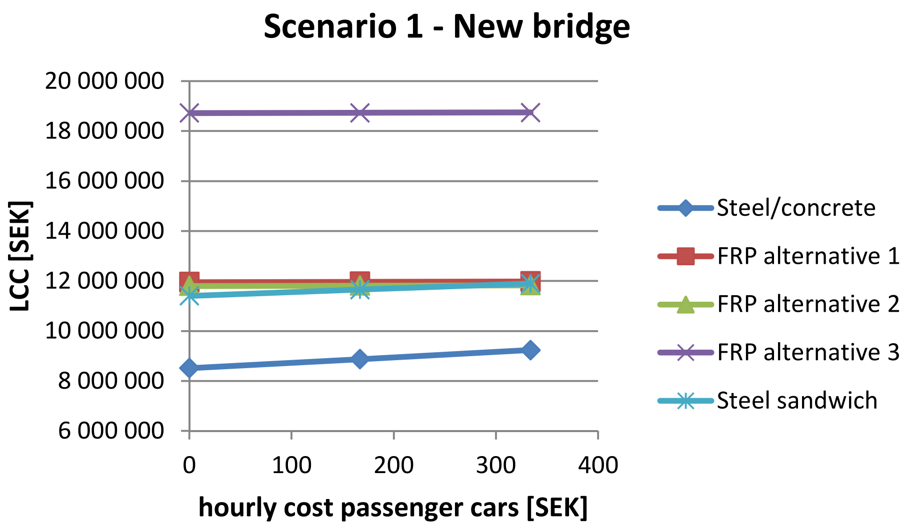

Figure 6.12. Effect of varying hourly costs, scenario 1.

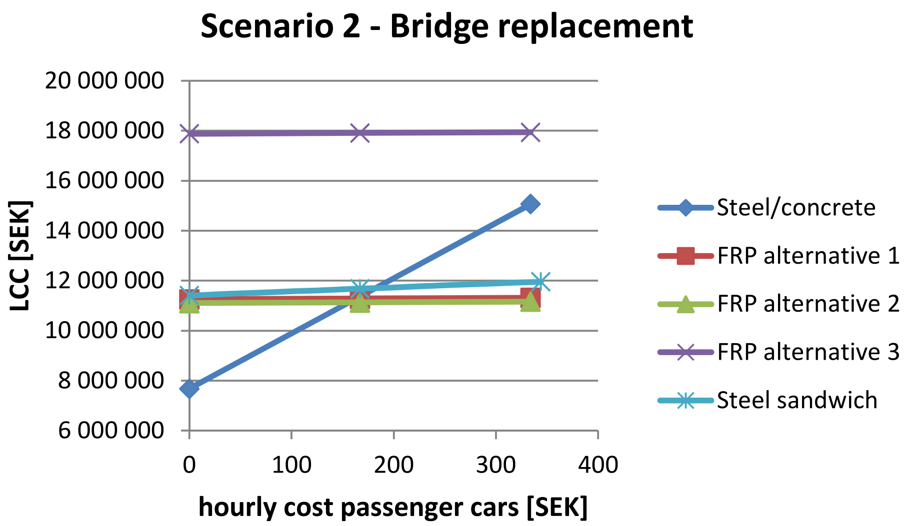

Figure 6.13. Effect of varying hourly costs, scenario 2.

Replacement of edge beams

How the total LCC is affected by a changed interval for the replacement of edge beams is shown in Figure 6.14 and Figure 6.15. Since there are no concrete edge beams for the FRP alternatives and the steel sandwich alternative, their LCCs stay constant. The steel/concrete bridge has a difference in price of SEK 670,000 when the highest interval is compared to the lowest, but the ranking between the alternatives stays the same in scenario 1.

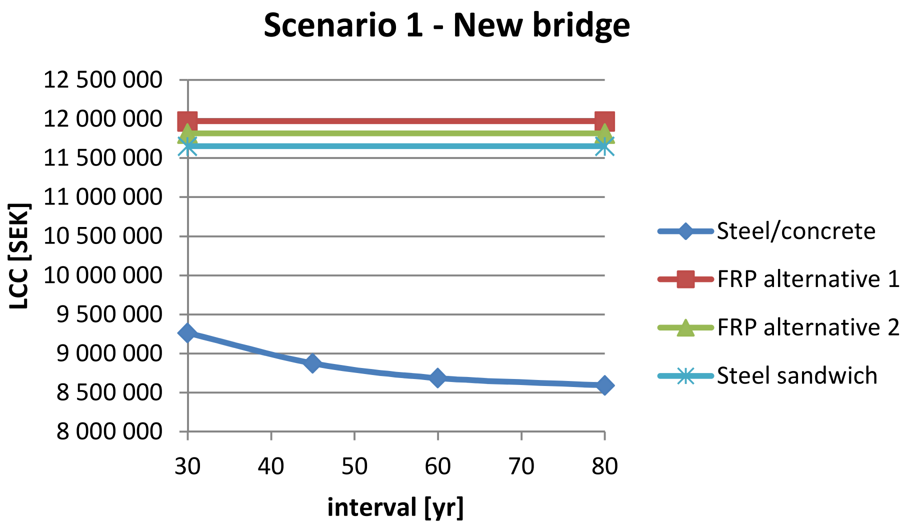

Figure 6.14. Effect of varying interval for replacement of edge beams, scenario 1. Note that FRP alternative 3 is not shown in the figure.

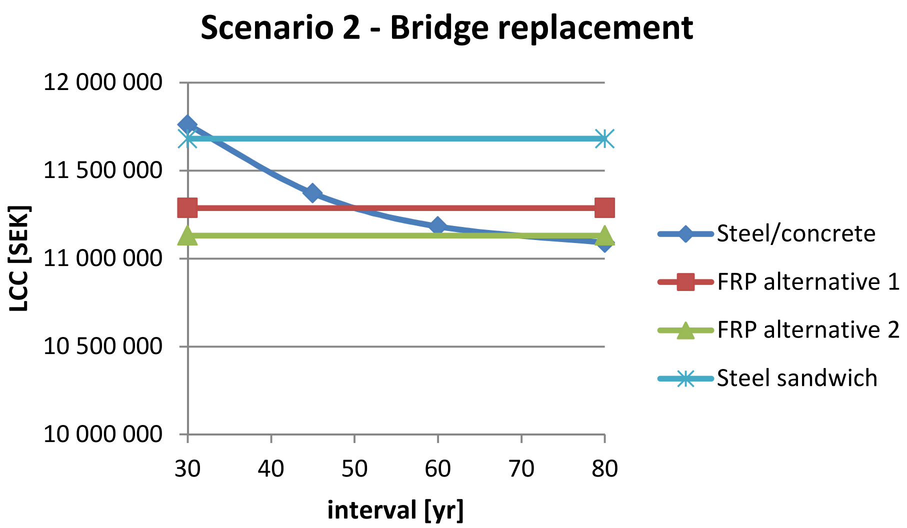

Figure 6.15. Effect of varying interval for replacement of edge beams, scenario 2. Note that FRP alternative 3 is not shown in the figure.

Reduced speed

The effect of the assumption of reduced speed was evaluated next. It showed to have a large influence on the total cost of the steel/concrete bridge in scenario 2 where the user costs compose a larger part of the cost, see Figure 6.17, while it did not affect the rank in scenario 1, see Figure 6.16. In scenario 2, the steel/concrete bridge has almost the same price as FRP alternative 1 and 2 and the steel sandwich alternative when the speed reduction is 5 km/h and 10 km/h but is the most expensive of all designs for a reduction of 30 km/h.

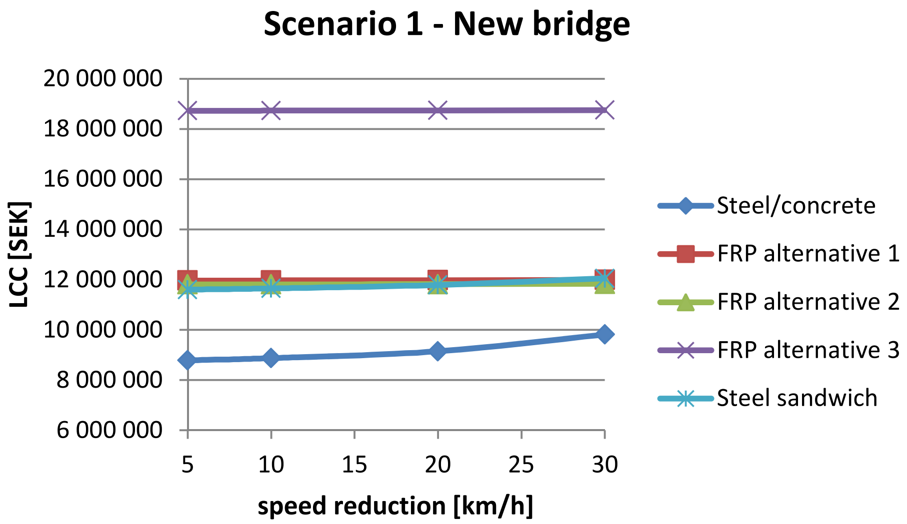

Figure 6.16. Effect of varying reduced speed, scenario 1.

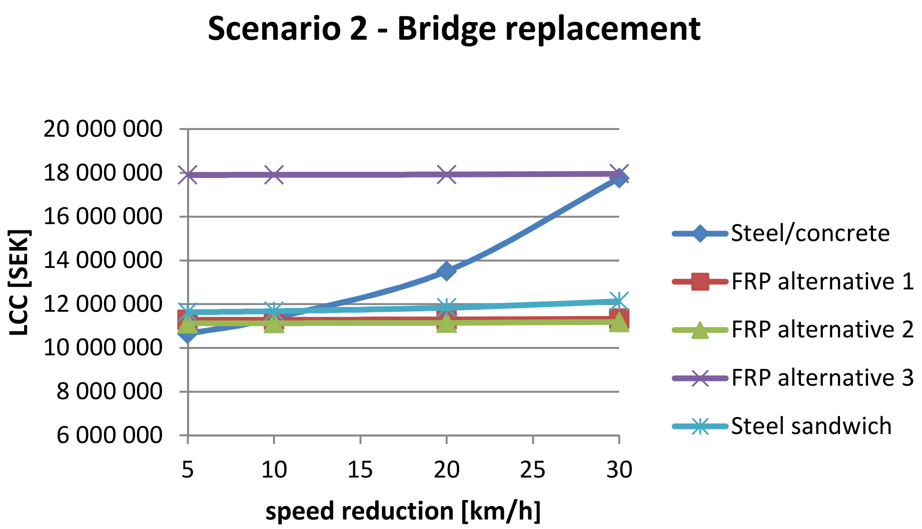

Figure 6.17. Effect of varying reduced speed, scenario 2.

Replacement of FRP deck

If the FRP deck would need to be replaced during the service life of the bridge, it would have a relatively small effect on the result, as can be seen in Figure 6.18 and Figure 6.19. The cost difference for replacement of the deck after 35 years compared to 80 years is around SEK 1.9 million for alternative 1 and 2.

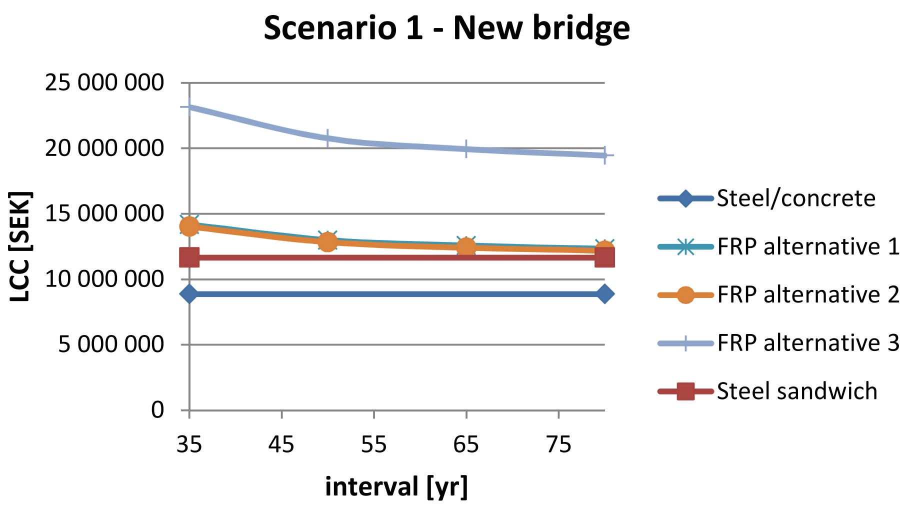

Figure 6.18. Effect of replacement of FRP deck, scenario 1.

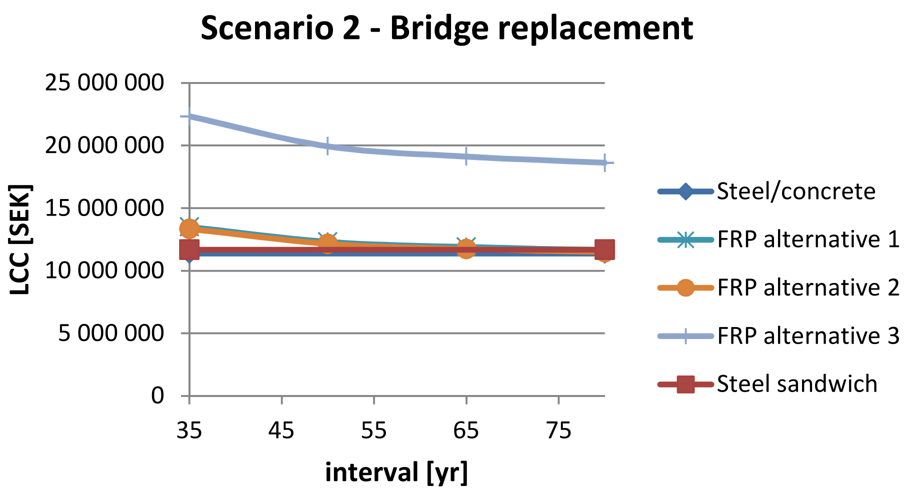

Figure 6.19. Effect of replacement of FRP deck, scenario 2.

Replacement of concrete deck

Lastly, a replacement of the concrete deck in the steel/concrete bridge was analysed. The interval was set to 50 years. This resulted in an increase of the LCC of approximately SEK 1 million for the steel/concrete alternative in both scenario 1 and 2. In scenario 1 the increased cost does not affect the ranking and the steel/concrete alternative still has the lowest LCC. However, in scenario 2 the increase in price makes the steel/concrete bridge somewhat more expensive than both FRP alternative 1 and 2 and the steel sandwich alternative.

6.2 Results from the life-cycle assessments

The emission vectors for FRP and polymer concrete were obtained through openLCA and used as input in BridgeLCA. These emission vectors are presented in Table 6.2 for 1 kg of FRP, 1 kg polyester based polymer concrete and 1 kg epoxy based polymer concrete respectively. In order to verify the emission vectors assembled by openLCA, the concrete material in BridgeLCA was used as a reference. The concrete, named *Concrete, normal, at plant/CH,* in ecoinvent was ran through openLCA and the resulting emission vector was compared to the one used in BridgeLCA. The emission vectors for the five categories GWP, ODP, AP, EP and FD, were identical in BridgeLCA and openLCA. However, there were large discrepancies for the toxicity categories due to the different versions of USEtox used by openLCA and BridgeLCA, see Appendix J. Therefore, the results of the toxicity impact categories are not evaluated further. As can be seen, the environmental impact of polymer concrete is quite similar regardless of the type of resin used.

Table 6.2. Emission vectors obtained with openLCA.

Impact category	Method	Unit	FRP	Polyester PC	Epoxy PC
GWP	ReCiPe (H)	kg CO2 eq	3,62E+00	1,49E+00	9,08E-01
ODP	ReCiPe (H)	kg CFC-11 eq	3,40E-07	1,54E-07	9,33E-09
EP	ReCiPe (H)	kg P eq	1,16E-03	3,40E-04	2,82E-05
AP	ReCiPe (H)	kg SO2 eq	1,49E-02	3,42E-03	5,22E-03
FD	ReCiPe (H)	kg oil eq	1,21E+00	5,10E-01	3,89E-01
ET	USEtox	CTUe	1,52E+00	4,97E-01	3,23E-01
HTC	USEtox	CTUh	1,68E-07	4,26E-08	5,11E-08
HTNC	USEtox	CTUh	7,21E-07	1,11E-07	1,16E-07

The results of the LCAs at midpoint level are presented in Figure 6.22 for five of the eight impact categories: GWP, ODP, EP, AP and FD. Moreover, a comparison of the final, normalized results is shown in Figure 6.21. The normalized results are presented in the unit person equivalents, that is one person equivalent corresponds to the environmental impact of one person per year.

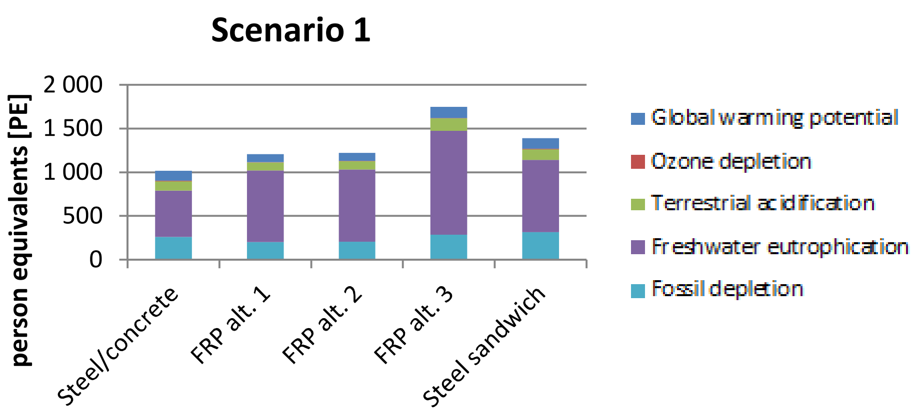

Figure 6.20. Comparison of normalized results for all alternatives, scenario 1.

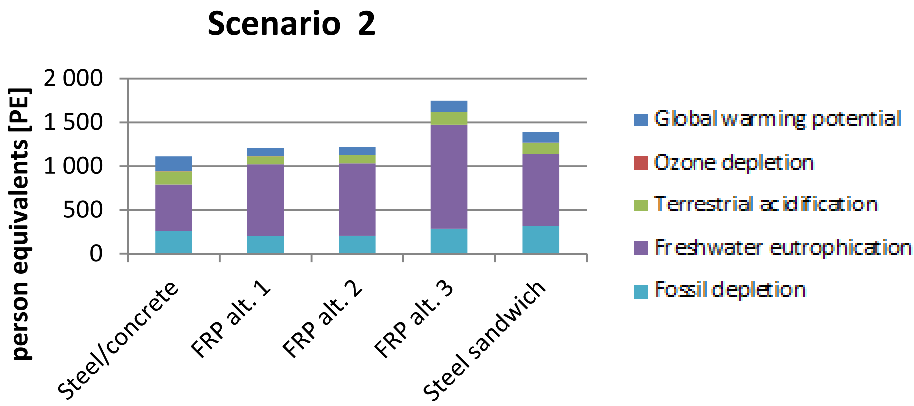

Figure 6.21. Comparison of normalized results for all alternatives, scenario 2

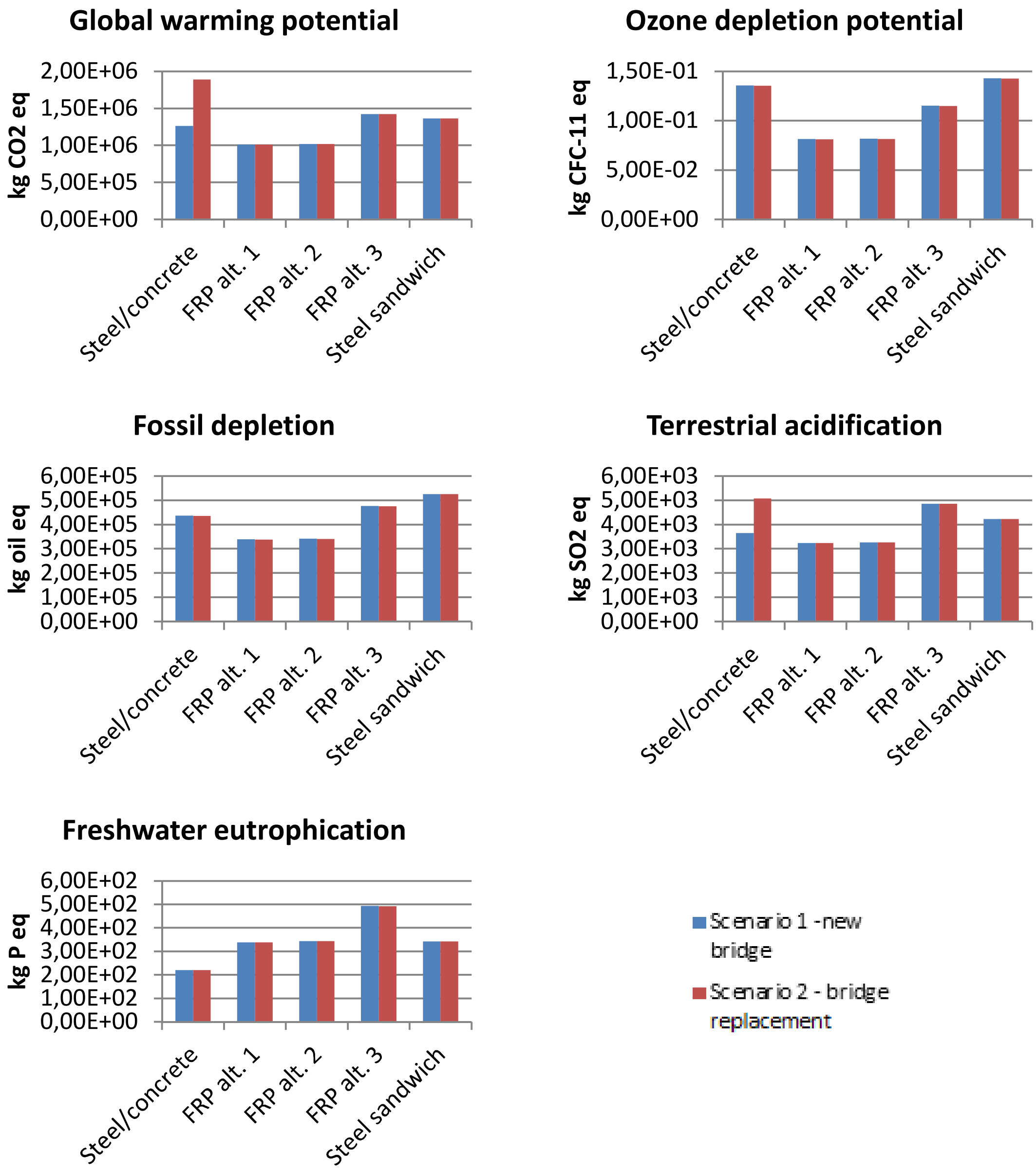

Figure 6.22. Midpoint results shown by impact category

The total normalized result shows that the steel/concrete bridge causes the least amount of person equivalents, followed by FRP alternative 1 and 2. The difference between scenaro 1 and 2 is negligible for all alternatives except the steel/concrete bridge.

Broken down category by category at midpoint level, it can be seen that FRP alternative 1 and 2 cause lower emission than the other alternatives in all impact categories except freshwater eutrophication. The only difference between scenario 1 and 2 is in the categories global warming potential and terrestrial acidification. All results form the LCA analyses can be found in Appendix K.

The weighted results are shown in Figure 6.23. It is evident that the chosen set of weighting has a large influence on the outcome of the analysis, especially in scenario 2. The most deviating result is received for the US-EPA method in scenario 2 where the steel/concrete bridge is clearly disfavoured due to the weighted importance of the global warming potential.

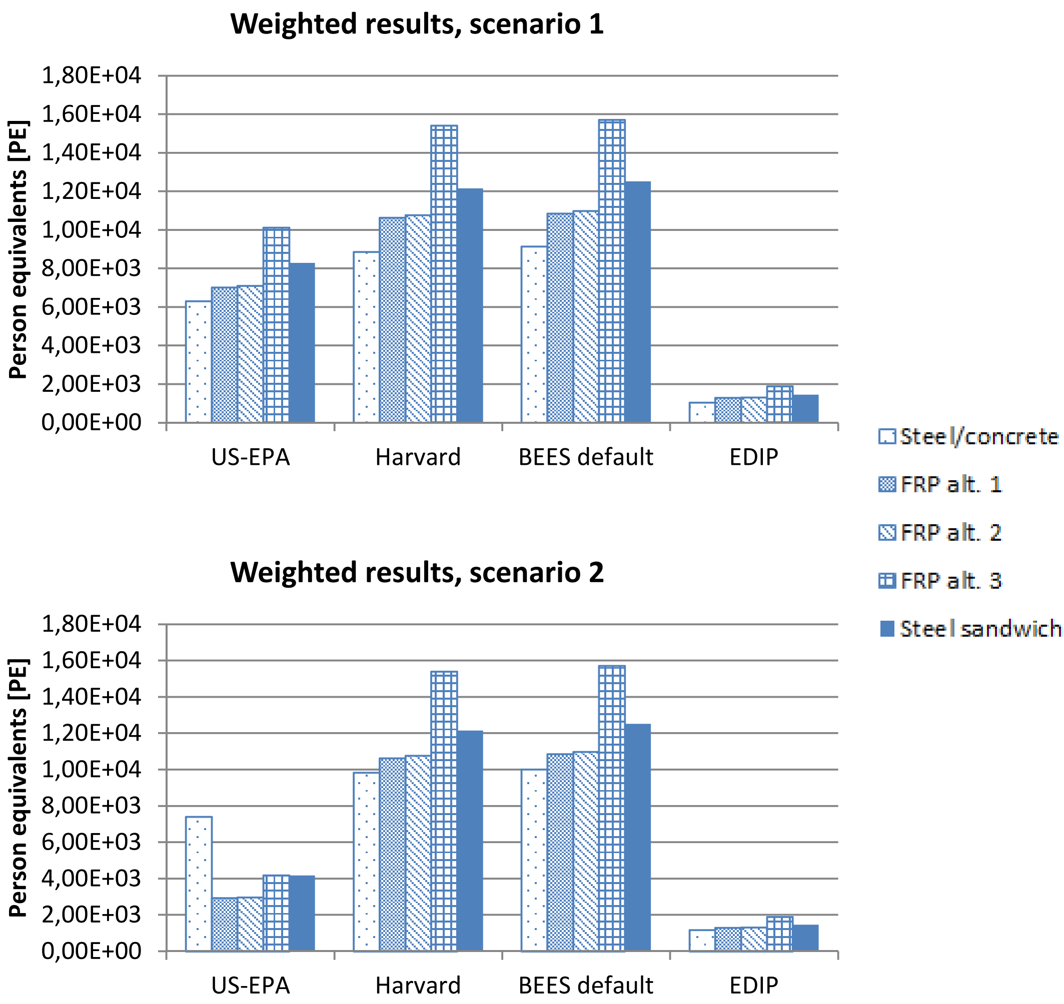

Figure 6.23. Weighted results.

7 Discussion and conclusions

In the case study performed in this thesis, a conventional steel/concrete bridge concept was compared to three different concepts using FRP bridge deck and one concept with a steel sandwich deck in terms of life-cycle cost and environmental impact. FRP alternative 3 was included in the case study in order to evaluate a concept with double FRP decks intended to increase the stiffness of the deck and reduce the dimensions of the steel girders and the exposed steel area. However, the preliminary design showed that more steel was still needed for FRP alternative 3. Thus, FRP alternative 3 used both more steel and more FRP than the other two FRP alternatives, giving higher costs and emissions throughout the results. Therefore, FRP alternative 3 is considered an unsuitable solution and will not be discussed further.

The alternatives were evaluated in two different situations. In scenario 1 a bridge is built on a new site where no traffic is affected by the construction. In scenario 2 the bridge is replacing an old structure, thus disrupting existing traffic flows.

Life-cycle cost analysis

The result shows that the user cost can be decisive in the decision-making process. Since these costs are indirect, it can be questioned whether the investor would really take them into account when choosing which bridge design should be built. However, in infrastructure projects the investor is often a government agency, such as Trafikverket, who therefore probably has larger interest in reducing the traffic disturbances than private investors. Moreover, governmental agencies have to take political decisions regarding for example sustainability and accessibility into account in the tendering phase.

As can be seen in the results of the sensitivity analysis, changes in discount rate have a large influence on the LCC of all alternatives. The higher the discount rate, the more influential are costs occurring in the investment phase and early operation phase compared to costs appearing later on in the service life. An argument for lowering the discount rate is that all costs during the service life would be more equal, thus enhancing the life-cycle perspective. With a high discount rate, costs are postponed to future generations which are not compatible with a sustainable development. On the other hand, future costs are always predictions while present costs will have to be paid. A majority of the costs for the FRP alternatives occur in the investment phase while the steel/concrete alternative and steel sandwich alternative have larger needs and costs for maintenance. Hence, a lower discount rate benefits the FRP alternatives. Since the discount rate used in Swedish infrastructure projects is set by Trafikverket, their decision on decreasing or increasing it will affect the outcome of LCCAs for projects like this case study. The current trend is that the recommended discount rate has been decreased over the last decades.

When it comes to the prefabricated FRP deck elements used in this case study, the material cost of the FRP forms a large portion of the LCC. We believe that the price of FRP will decrease as the produced volume increases, affecting the outcome of similar studies in the future. However, in the specific situation of this case study the price would have to be decreased more than 50% in order for the FRP bridges to be profitable in scenario 1, construction of a new bridge. Such a large price reduction does not seem realistic in a near future.

Regarding the end-of-life phase, the costs form less than 1 % of the total cost using the net present value method. One conclusion is that, as long as the discount rate is set as high as 3.5 %, the end-of-life costs are negligible in projects with long service lives, such as bridges.

By comparing the results of the scenario 1 and 2, it is evident that the traffic volume affected in the investment phase has a large influence on the LCC. If a large traffic volume is affected by traffic disruptions, the user cost will increase accordingly. The effect of the parameters reduced traffic speed and hourly user cost also increases with increased traffic volumes. The effect of an increased traffic volume can be seen both in the results from the sensitivity analysis and in the comparison between scenario 1 and 2. Since the construction of the FRP alternatives and the steel sandwich alternative affect small volumes of traffic in both scenarios, due to their fast erection, their LCCs hardly differ between the two scenarios. The steel/concrete bridge though, affects a substantially larger traffic volume in scenario 2 than in scenario 1 and the portion of user costs for therefore increases from 4% to 29% and the LCC with over SEK 3 million. Consequently, in a competition between steel/concrete, steel sandwich and FRP, the steel/concrete concept is disfavoured by the traffic situation in scenario 2. It is also clear that the user costs arising from increased traffic volumes favour new structural solutions with prefabricated components that can be assembled quickly on site.

The overall conclusion from the economic part of the case study is that, in this specific situation, a conventional steel/concrete bridge has a lower LCC than any of the innovative steel sandwich and FRP alternatives in scenario 1. In specific, the difference in price between the steel/concrete bridge and the FRP alternatives is around SEK 3 million, corresponding to roughly 25%. It should be noted though that when the common costs for foundation, abutments etc. is added, the relative difference of the total cost between the alternatives will decrease. In scenario 2, the difference in LCC is very small with a slight advantage for FRP alternative 1 and 2. Furthermore, there are fewer common costs in this scenario since only the superstructure is replaced and therefore the relative difference in price is more just.

Since all case studies handle specific situations the conclusions drawn here cannot be directly applied to another bridge construction project. However, the results of the sensitivity analysis suggest that a conventional alternative with low production cost and longer production time is more profitable as long as the traffic volume affected is low while a prefabricated alternative with a higher investment cost but a rapid construction time should be considered in more complex traffic situations. In particular, the combined effect of a busy traffic situation, decreasing FRP price and a low discount rate would definitely benefit the FRP alternatives.

Lastly, there are some uncertainties in the case study which should be taken into account when drawing conclusions and using the case study as a base for decisions. Future maintenance and operation costs are predictions based on experience and there exist uncertainties regarding when these costs will occur, how time consuming and how large they will be or even if they will occur. Moreover, it is possible, though unlikely, that the bridge will not be needed for the intended service life or that the traffic context will change. Second, parameters concerning the indirect user costs, such as the time value, are subjective. Third, the assumptions regarding reduced speed, traffic disruptions and detours are quite rough in this case study. These parameters are hard to estimate but more certain predictions could be made if traffic simulations were incorporated in the case study. This could be recommended especially in complex traffic situations.

Regarding the steel sandwich concept the uncertainties in the analysis are larger than for the other alternatives since it is still on a conceptual level and a number of issues, such as acoustics and vibrations, remain to be solved. Moreover, the steel price used throughout the analyses is for steel girder material and might not be accurate for the large plates used for the sandwich deck. There are also reservations regarding corrosive damage to the steel. With large amounts of exposed steel area these damages can be hard to prevent and costly to repair.

Life-cycle assessment

The results from the LCA showed that the steel/concrete bridge had the lowest total impact of the analysed designs. However, all categories except freshwater eutrophication favoured FRP alternatives 1 and 2 when studied individually at midpoint level. When the midpoint results from the LCA were normalized, the impact category freshwater eutrophication was scaled up, thus composing a large portion of the total impact, whereas for example ozone depletion was scaled down to a negligible amount. This implies that the normalization factors have a large impact on the total result and that they should be chosen with care in order to get fair results.

The weighted results can have different outcome, depending on the set of weighting factors used. For example, when the US-EPA method was used the steel/concrete bridge suddenly had a substantially higher environmental impact than the other alternatives in scenario 2. This shows that if weighting is included in the analysis it must be clearly stated which method has been used, and how the different impact categories have been rated and why.

In some previous studies, the environmental impact of FRP bridges has been compared to other bridge types by taking only carbon emissions into account. The result from this case study shows that such a comparison can be quite misleading since carbon emissions are not dominating the total environmental impact. The normalized result indicates that a simplified LCA analysis, considering only one impact category, should consider freshwater eutrophication rather than carbon emissions.

When comparing the results of scenario 1 and 2, the only difference was the environmental impact from global warming potential and terrestrial acidification. This is due to the extra traffic emissions in scenario 2. For the steel/concrete bridge, the environmental impact in these categories was substantially higher in scenario 2 than scenario 1. However, this change was not enough to increase the total environmental impact of the steel/concrete bridge above the total impact of FRP alternative 1 and 2. In situations with larger traffic volumes, it is possible that the increased impact for the steel/concrete bridge would motivate a choice of FRP alternative 1 or 2.

BridgeLCA was used to compute the LCA in this case study. Since this tool is developed mainly for performing LCAs of bridges using traditional materials, there were limitations regarding the possibilities to include unconventional materials like FRP and polymer concrete. First of all, it was not possible to consider transportation of these materials from the factory to the site, leading to that such transportations had to be excluded for all materials in order to get fair results. Considering that previous studies have shown that transportations in the construction phase can have great influence on the total result, it is possible that the results would have been different if these transports could have been included. Since the steel/concrete bridge requires larger cranes and more material transports than the other alternatives, it would probably be the most disfavoured alternative if transports had been included.

Secondly, the environmental impact vectors for the unconventional materials had to be established and inserted in BridgeLCA manually by the user. Because of the authors' limited experience in developing such impact vectors, several uncertainties were introduced at this stage and it was difficult to review the reliability of the results. For example, when developing the impact vector for polymer concrete, a predefined process from ecoinvent for the polyester resin was chosen, rather than specifying each component in the resin ourselves. This leads to that all relevant steps in the production process are taken into account, but the ingredients in the resin might not be exactly as in the actual polymer concrete. The uncertainties that these assumptions entail should be kept in mind when analysing the results.

The toxicity impact categories were excluded from the results in this case study since there is no recommended normalisation for these categories and the scientific base is questioned. For future studies, it would be desirable to develop the methods for calculating the toxicity impact in order to achieve more complete results.

Appraisal of the life-cycle assessment and the life-cycle cost results and final conclusions

Life-cycle cost analyses and life-cycle assessments can be evaluated separately or together by integration. A number of integration methods were described in Chapter 4. In general, these methods are still on a theoretical level and they always require subjective choices with regard to how environmental impacts are assessed compared to costs. Some of the integration methods include advanced probabilistic studies, which decrease the subjectivity but also makes the integration more complicated to perform. Therefore, it was chosen to evaluate the results of the life-cycle cost analyses and life-cycle assessments in the case study separately and base the final decision on the credibility of the analyses.

To carry out an LCC analysis is quite straight-forward and can most likely be done with the skills of a civil engineer. An LCA analysis on the other hand, is complex and requires much knowledge in the environmental area in order to evaluate which processes to include and the reliability of the results. Our assessment is therefore that there are more uncertainties in the LCA results than the LCC results of the case study in this thesis. Based on this, more weight was put on the results of the LCC than the LCA when recommending a bridge design from the case study.

In scenario 1, the cost of the steel/concrete bridge is considerably lower than the FRP designs and the results from the LCA are also in favour of the steel/concrete bridge. Therefore, it can be concluded that a conventional steel/concrete bridge is preferable in this scenario.

In the bridge replacement scenario, both cost and environmental impact are very similar between the steel/concrete bridge, the steel sandwich concept and FRP alternative 1 and 2. The LCC result slightly favours the FRP designs whereas there is a slight advantage for the steel/concrete bridge regarding environmental impact. Therefore, the decision on which design to choose depends on if the cost or the environmental impact is prioritised.

The differences in LCC between the four alternatives are so small, all in a range of SEK 500,000, that they are in the margin of error. Therefore, there is no obvious winner among the alternatives in scenario 2. With regard to the LCA results though, the steel sandwich bridge fell short compared to the steel concrete bridge, FRP alternative 1 and FRP alternative 2. In the choice between the three remaining designs, the decision can be taken considering the possibilities and limitations of the proposed solutions.

When it comes to the steel/concrete design, it is a traditional solution which the industry has much knowledge and experience in. With this follows that the risks are smaller but also that the cost-effective limit is nearly reached.

FRP alternative 1 and 2 show similar results, economically and environmentally, as the steel/concrete bridge in scenario 2, thus FRP is still new in the bridge industry but is already competitive in this case study. Furthermore, there are great possibilities for optimizing the use of FRP, with regard to both price and environmental impact, in bridge applications. Choosing one of the FRP alternatives can contribute to this development of FRP concepts in the bridge industry. Hence, more sustainable bridge construction can be achieved.

In the choice between FRP alternative 1 and 2, the cost differs with around SEK 150,000 in favour of alternative 2, and the environmental impact differs with 15 person equivalents in favour of alternative 1. These differences are within the margin of error and therefore the decision should be based on which solution is most feasible technically.

The steel sandwich solution has the potential to become economically competitive on the market due to its light-weight, high strength and fast assembly. However, it needs to be further developed since many structural details remain unsolved. This analysis does not give a complete picture of the steel sandwich alternative but an indication of future possibilities.

8 Recommendations for future studies

During the work of this thesis, a number of interesting questions arose, which could be answered by further studies in the area.

Regarding the case study, it would be interesting to examine bridges set in different traffic situations to see how that affects the user costs. Moreover, a case where the light-weight of the FRP and steel could make it possible to avoid mid supports could be evaluated. It would also be interesting to compare bridges with FRP and steel decks to designs with prefabricated concrete decks.

This case study has focused on the overall evaluation of the different bridge concepts, further investigations could focus on more specific areas such as detailed designs or a certain phase of the life-cycle. In that case, costs and environmental impacts from machinery, transports etc. during the construction phase would be of special interest since that was omitted in this thesis.

Regarding the input for this thesis, there is a lack of data, especially concerning FRP and polymer concrete. To gather these data and incorporate it in databases would be an important part of further research in the area.

On a more general level, sensitivity analysis could be performed for LCA as well as for the LCCA. The sensitivity analysis can favourably be made by implementing Monte Carlo simulations to put the results in perspective. It would also be useful to develop the social aspect of sustainability, and methods to incorporate it with LCC and LCA results, in future studies.

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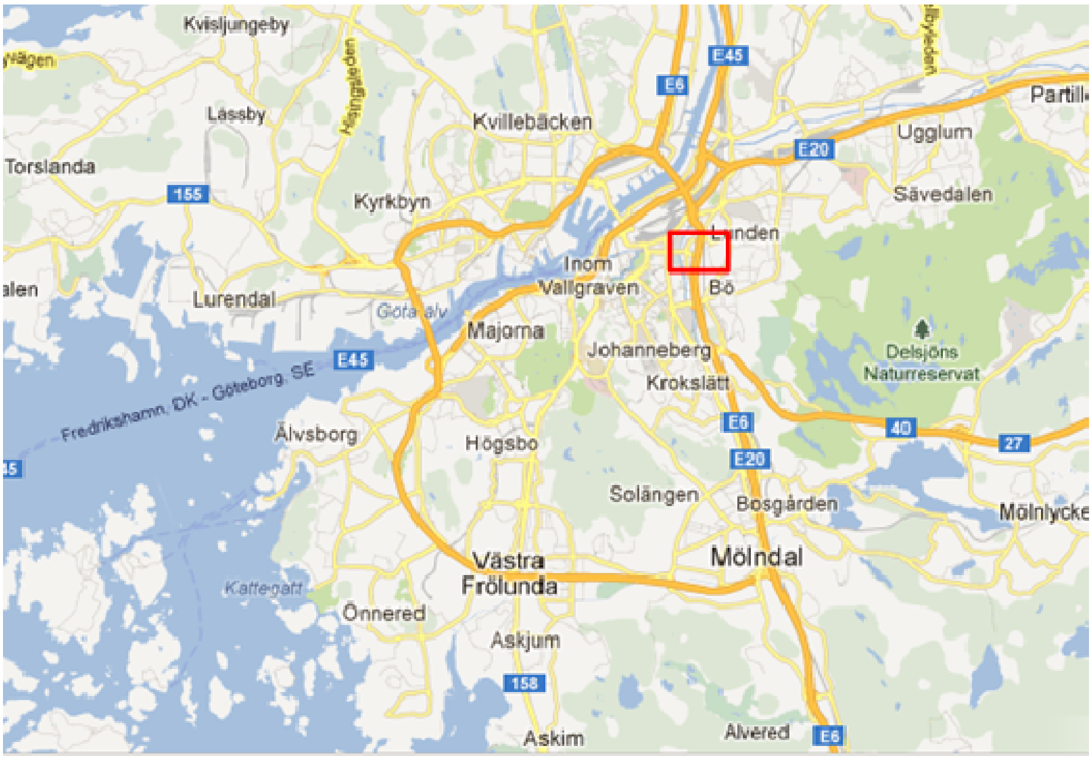
Appendices

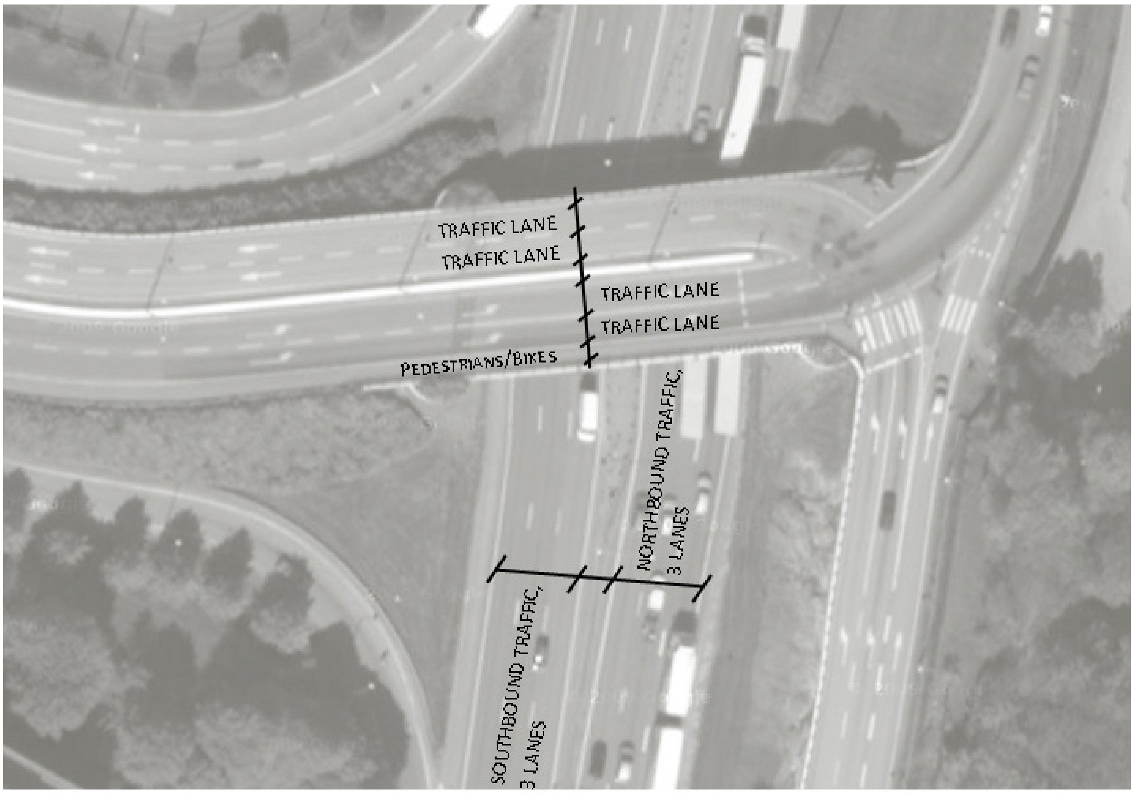
Appendix A

Bridge location and traffic layout

Location of the bridge, the upper map is an enlargement of the marked area in the overview of Göteborg below. Maps from Google maps and Eniro kartor.



The traffic layout on the bridge at Ullevimotet and on E6. On the bridge there are in total 4 traffic lanes, 2 in each direction, and 1 pedestrian lane. At E6 there are 3 lanes in each direction.

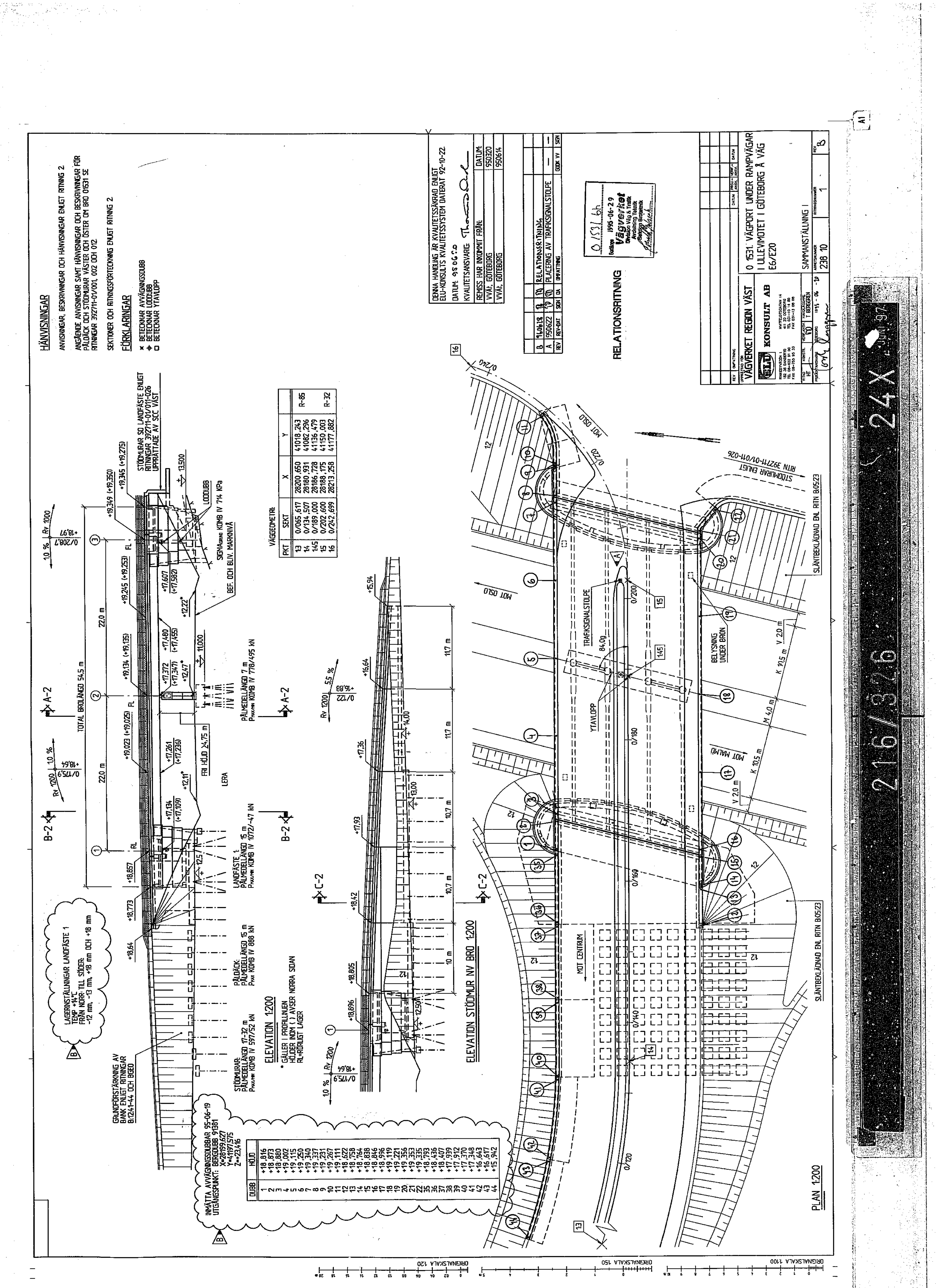


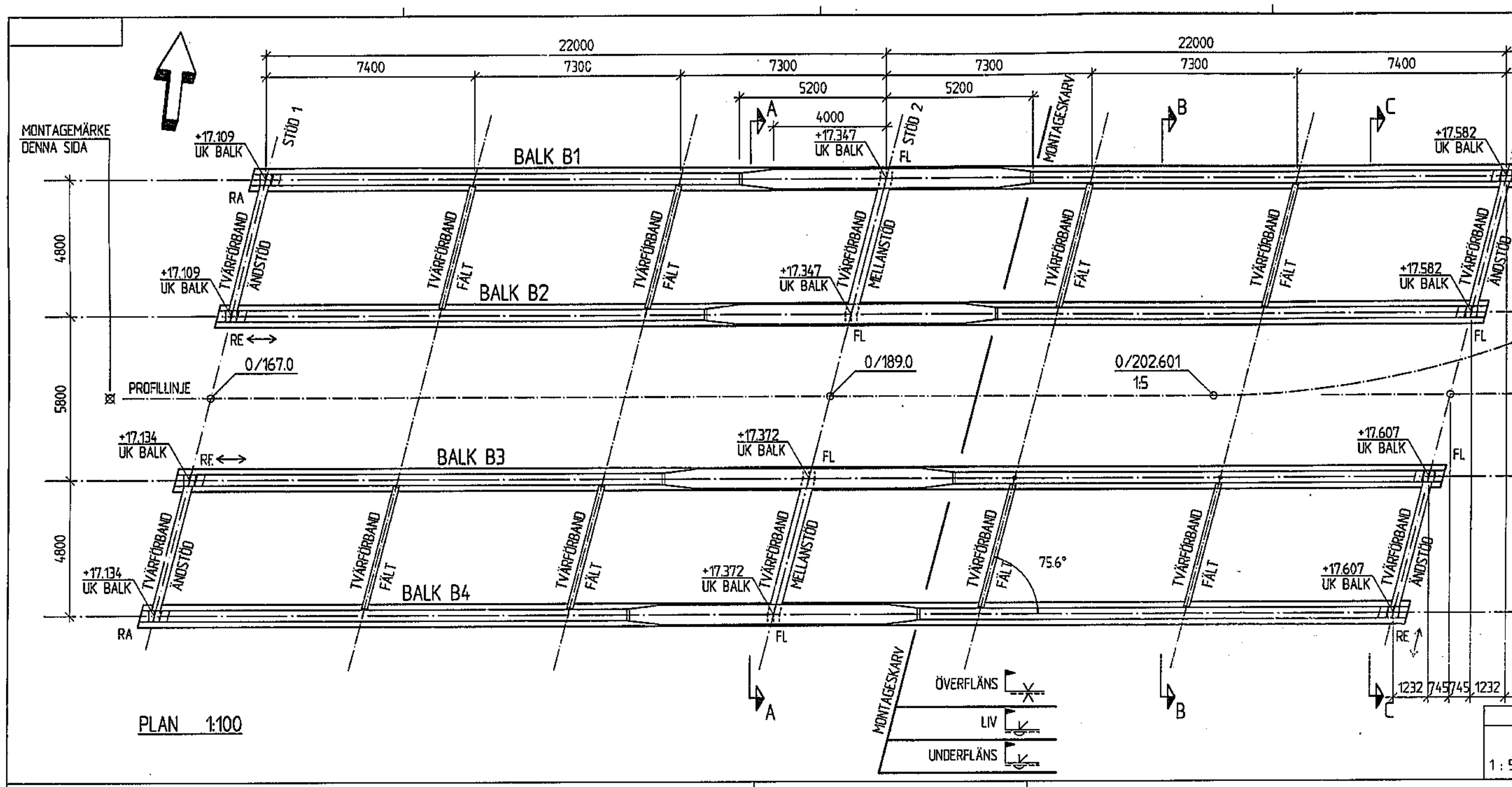
Appendix B

Drawings of bridge 14-1531-1 Ullevimotet

The first drawing is an assembly showing the elevation and plan of the whole bridge. The second is an assembly drawing of the steel components of the bridge.

Drawings are from Trafikverekts' database BaTMan.

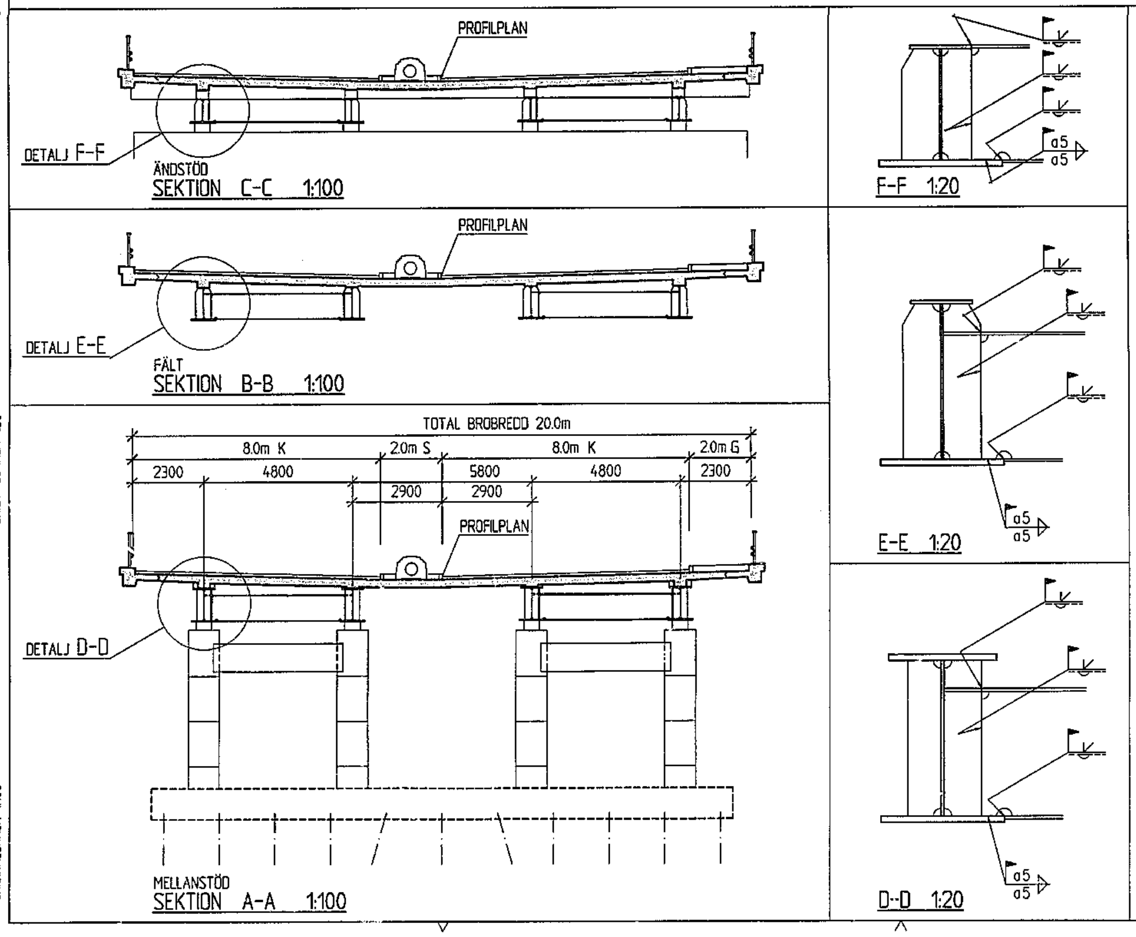

RELATIONSRITNING
VÄGVERKET REGION VÄST
0 1531 VÄGPORT UNDER RAMPVÄGAR I ULLEVIMOTET I GÖTEBORG Å VÄG E6/E20
ELEVATION 1:200
ELEVATION STÖDMUR NV BRO 1:200
PLAN 1:200
HÄNVISNINGAR
FÖRKLARINGAR



FL = FAST LAGER
RA = ALLSIDIGT RÖRLIGT LAGER
RE = ENSIDIGT RÖRLIGT LAGER
↔ ANGER RÖRELSERIKTNING

KOORDINATER

	PKT	X	Y
	0/167.0	28184.387	41114. 604
	0/189.0	186.728	136. 479
1 : 5	0/202.601	188. 175	150. 003



PÅ RITNING ANGIVNA MÅTT GÄLLER VID 20° C.
TOLERANSER: UTÖVER KRAV I BRONORM 88 GÄLLER ÄVEN TOLERANSKRAV ENL SBI PUBL. 91

HÅLTAGNING OCH SVETSNING I BÄRANDE STÅLDETALJER UTÖVER VAD SOM REDOVISAS PÅ RITNINGAR, FÅR EJ SKE UTAN KONTAKT MED KONSTRUKTÖREN.

REPRESENTANT FRÅN VÄGVERKET SKALL BEREDAS TILLFÄLLE ATT NÄRVARA VID TILLVERKNINGEN I VERKSTAD.

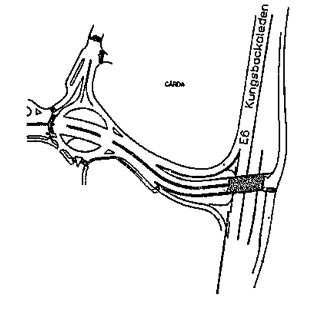
MONTAGEFÖRFARANDE

VÄSTRA DELEN AV BALK B1 (RESP B2, B3 OCH B4) LÄGGS UPP PÅ LAGREN STÖD 1 OCH STÖD 2. ÖSTRA DELEN AV BALK B1 (RESP B2, B3 OCH B4) LÄGGS UPP PÅ LAGER STÖD 3 OCH MONTAGESVETSAS (CA 5.2m ÖSTER OM STÖD 2) TILL VÄSTRA DELEN AV BALK B1 (RESP B2, B3 OCH B4) MONTAGESVETSEN SKALL VARA KOMPLETT INNAN BALKARNA FÅR BELASTAS AV FORM FÖR BETONGFARBANA.
TVÄRBALKARNA MONTERAS OCH MONTAGESVETSAS TILL RESP. BALK.

REV	REV-DAT	SIGN	QA	OMFATTNING	GODK VV	SIGN
A	960628			RELATIONSRITNING		

RELATIONSRITNING

RITNINGSFÖRTECKNING STÅLÖVERBYGGNAD
21 SAMMANSTÄLLNING STÅL
22 PRIMÄRBALK B1 OCH B3
23 PRIMÄRBALK B1 OCH B3 DETALJER
24 PRIMÄRBALK B2 OCH B4
25 PRIMÄRBALK B2 OCH B4 DETALJER
26 TVÄRBALKAR



ANVISNINGAR STÅL

ARBETET UTFÖRS ENLIGT BRONORM 88, (PUBL 1988:201-202, 1993:203, 1988:204-207, 1993:208), OCH TILLHÖRANDE BROBREV NR 6 (PUBL. 1993:1) SAMT ENLIGT STÅLBESTÄMMELSER BSK-1987

SVETSNING UTFÖRS ENLIGT AV VÄGVERKET STÄMPLAD SVETSPLAN. SVETSNING FÅR ENDAST UTFÖRAS AV SVETSARE SOM AVLAGT GODKÄNDA PROV ENLIGT SS 06 52 01 OCH SS 06 53 01.

ARBETET UTFÖRS I UTFÖRANDEKLASS GA. ENLIGT BRONORM 88 KAP 55, SAMT STÅLBESTÄMMELSER BSK-1987 KAP. 8

BRON HÄNFÖRS TILL MILJÖKLASS M3 ENLIGT BRONORM 88 KAP 31.11 SAMT BSK-1987 KAP 1:23.

UTÖVER NORMENLIG GRUNDKONTROLL ENLIGT BSK 9:4 OCH BRONORM 88 KAP 56 SKALL TILLÄGGSKONTROLL UTFÖRAS ENLIGT BRONORM 88 KAP 56.5 SAMT ENLIGT AV VÄGVERKET STÄMPLAD KONTROLLPLAN

SVETSNING ENLIGT BRONORM 88 KAP 55.2. SVETSAR UTFÖRS ENLIGT SVETSKLASS WB-K OM INTE ANNAT ANGES

MÅLNINGSSYSTEM UTFÖRS ENL BRONORM 88 KAP 55.32
TÄCKSKICKTETS KULÖR: ...

YTOR, SOM INTE KAN ROSTSKYDDSBEHANDLAS EFTER MONTERING GES FULLSTÄNDIG ROSTSKYDDSBEHANDLING FÖRE MONTERING. YTOR SOM BERÖRS AV SVETSNING MÅLAS FÖRST SEDAN SVETSNINGEN UTFÖRTS.

BÄTTRINGSMÅLNING UTFÖRS [illegible] BRONORM 88 KAP 72.3

STÅLYTOR MOT BETONG MÅLAS [illegible], GRUND OCH MELLANFÄRG INDRAS DOCK 50mm FRÅN FRI KANT.

VID UTFÖRANDE AV SAMTLIGA STÅLDETALJER GÄLLER ATT ALLA SNITT OCH ALLA HÅL SKALL VARA JÄMNA OCH FRIA FRÅN GRADER.

GENERELLT GÄLLER ATT KANTER AVRUNDAS SÅ ATT RADIEN ÄR STÖRRE ELLER LIKA MED 3 mm ENLIGT BRONORM 88 KAP 55.314. NOTCHAR FASAS I 45° VINKEL TILL 1/3 AV GODSTJOCKLEKEN ENL TYPDETALJ PÅ RITNING.

INNAN LEVERANS FRÅN VERKSTADEN FÅR SKE, SKALL VÄGVERKETS REPRESENTANT BEREDAS TILLFÄLLE ATT BESIKTIGA STÅLDETALJERNA.

MATERIAL ENLIGT BRONORM 88 KAP 54.1
STÅLMATERIAL: SS 2134 OM EJ ANNAT ANGES PÅ RITNINGARNA.

SVETSBULTAR ENLIGT BRONORM 88 KAP 54.3

ELEKTRODER ENLIGT BRONORM 88 KAP 55.2.

FINKORNSTÅL MED GODSTJOCKLEK STÖRRE ÄN 25mm LEVERERAS NORMALISERAT ENLIGT BRONORM 88 KAP 54.1

SAMTLIGA PLÅTMÅTT ÄR TEORETISKA "KANT-I-KANT" MÅTT. HÄNSYN ÄR EJ TAGEN TILL FOGBEREDNING. VERKSTADEN VÄLJER SVETSSPALT OCH SVETSNÄSA SAMT ÖVERMÅTT FÖR SVETSKRYMPNING.

TVÄRSNITTSAREAN FÖR STÅLDETALJERNA FÅR INNEHÅLLA EN MINUSAVVIKELSE AV HÖGST 6% AV NOMINELL AREA.

DENNA HANDLING ÄR KVALITETSSÄKRAD ENLIGT ELU-KONSULTS KVALITETSSYSTEM DATERAT 92-10-22.
DATUM: 941014
KVALITETSANSVARIG: [signature]

REMISS HAR INKOMMIT FRÅN:	DATUM:
VVÄt, GÖTEBORG	94-09-28

6 1531 e
Bofilage 1994-10-21
Vägverket
Division Väg & Trafik
Avdelning Teknik
Sektion Broteknik

REV	OMFATTNING	DATUM	PROJ.-ANSV.	VERIF.-ANSV.	DATUM

UPPRÄTTAD FÖR
VÄGVERKET REGION VÄST

ELU KONSULT AB
RINKEBYVÄGEN 1, 182 36 DANDERYD, TEL 08-622 91 00, FAX 08-755 95 33
HVITFELDTSGATAN 14, 411 20 GÖTEBORG, TEL 031-10 14 80, FAX 031-13 08 06

RITAD	KONSTR.	VERIF.ANSV.	HANDLÄGGARE
KB	KM		T BERGGREN

PROJEKTANSVARIG GÖTEBORG 1994-10-14

O 1531 VÄGPORT UNDER RAMPVÄGAR I ULLEVIMOTET I GÖTEBORG Å VÄG E6 / E20
STÅLÖVERBYGGNAD
SAMMANSTÄLLNING STÅL

ARBETSNUMMER	RITNINGSNUMMER	REV
23810	21	A

A1

216/267 24X

Appendix C

Material amounts - steel/concrete bridge at Ullevi

All geometries in this document are found throguh the drawings of bridge *14-1531-1 Ullevimotet* available in Trafikverket's database BaTMan. Approximations and average dimensions have been used throughout.

Geometries bridge deck

$l_{tot} := 54.5m$

$b_{gc} := 2m$

$l_{span} := 22m$

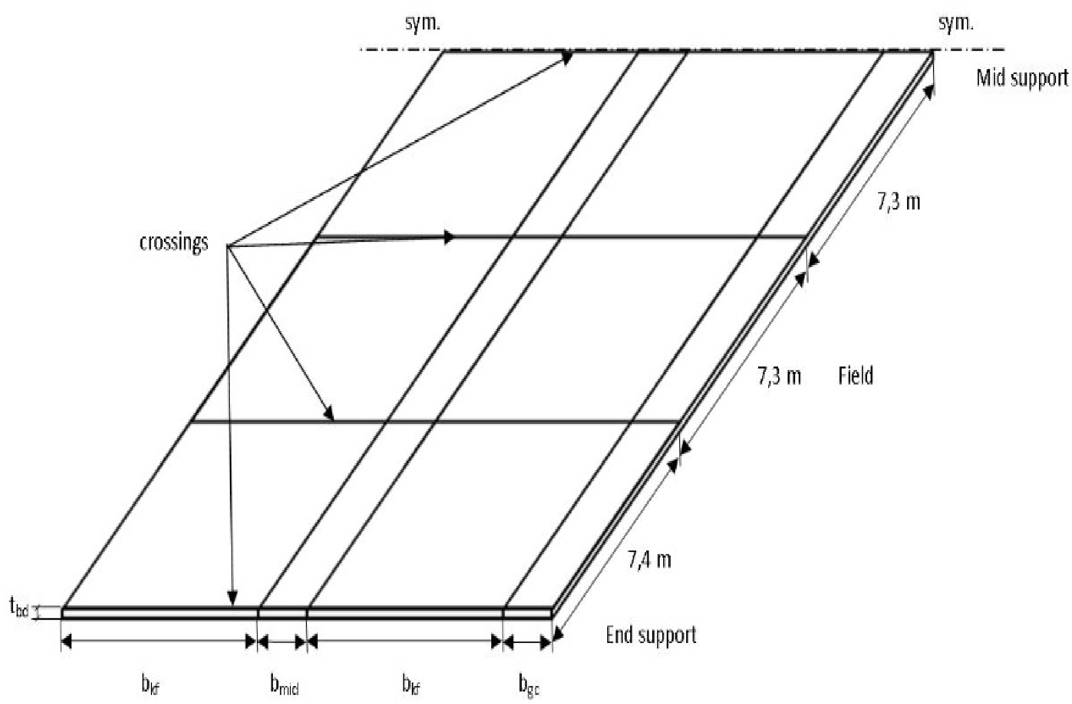
$b_{kf} := 8m$

$n_{span} := 2$

$b_{mid} := 2m$

$b_{tot} := 20m$

$t_{bd} := 0.26m$



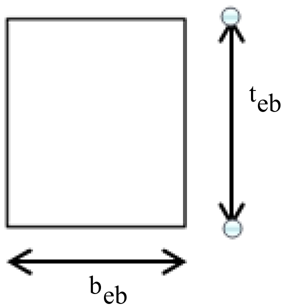
Geometries edge beams

$l_{eb} := 45m$

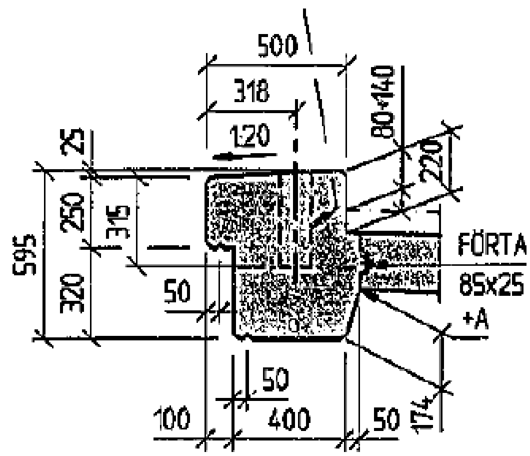
$b_{eb} := 0.5m$

$t_{eb} := 0.595m$

$n_{eb} := 2$



this geometry was approximated from -->



Geometries steel girders (average):

$h_{web} := 1000mm$

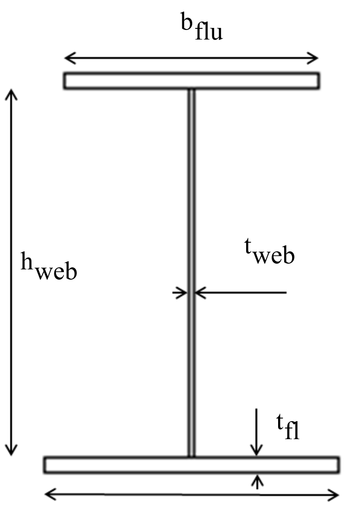
$t_{web} := 20mm$

$b_{fll} := 800mm$

$b_{flu} := 700mm$

$t_{fl} := 40mm$

$n_{gird} := 4$

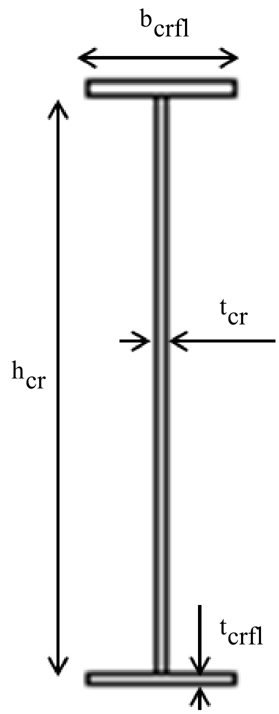


$$I := \frac{b_{flu} \cdot t_{fl}^3}{12} + \frac{b_{fll} \cdot t_{fl}^3}{12} + t_{web} \cdot \frac{h_{web}^3}{12} + b_{flu} \cdot t_{fl} \cdot \left(\frac{h_{web}}{2} + \frac{t_{fl}}{2}\right)^2 \ ... = 1.79 \times 10^{10} \cdot mm^4$$
$$+ b_{fll} \cdot t_{fl} \cdot \left(\frac{h_{web}}{2} + \frac{t_{fl}}{2}\right)^2$$

Geometries steel crossings, dimensions of the crossings differ in field, at end support and at mid support. An average was calculated.

Field:

General illustration:



$n_{crfield} := 8$

$l_{crfield} := 4438mm$

$t_{crfield} := 10mm$

$h_{crfield} := 800mm$

$t_{crfieldfl} := 15mm$

$b_{crfieldfl} := 200mm$

End support:

$n_{cresup} := 4$

$l_{cresup} := 4540mm$

$t_{cresup} := 15mm$

$h_{cresup} := 725mm$

$t_{cresupfl} := 15mm$

$b_{cresupfl} := 300mm$

Mid support:

$n_{crmsup} := 2$

$l_{crmsup} := 4433mm$

$t_{crmsup} := 15mm$

$h_{crmsup} := 800mm$

$t_{crmsupfl} := 20mm$

$b_{crmsupfl} := 400mm$

Densities

$$\rho_{con} := 2400 \frac{kg}{m^3}$$

$$\rho_{steel} := 7800 \frac{kg}{m^3}$$

$$\rho_{asp} := 2.38 \frac{kg}{1000cm^3}$$

Estimated value after number on ABT apshalt.

Amount of concrete

$$V_{eb} := l_{eb} \cdot b_{eb} \cdot t_{eb} \cdot n_{eb} = 26.775 \cdot m^3$$

$$V_{bd} := l_{tot} \cdot \left(b_{kf} \cdot 2 + b_{gc} + b_{mid}\right) \cdot t_{bd} = 283.4 \cdot m^3$$

$$m_{eb} := \rho_{con} \cdot V_{eb} = 64.26 \cdot tonne$$

$$m_{bd} := \rho_{con} \cdot V_{bd} = 680.16 \cdot tonne$$

$$m_{con} := m_{eb} + m_{bd} = 744.42 \cdot tonne$$

Amount of steel

$$V_{crfield} := l_{crfield} \cdot \left(t_{crfield} \cdot h_{crfield} + 2 \cdot t_{crfieldfl} \cdot b_{crfieldfl}\right) = 0.062 \cdot m^3$$

$$V_{cresup} := l_{cresup} \cdot \left(t_{cresup} \cdot h_{cresup} + 2 \cdot t_{cresupfl} \cdot b_{cresupfl}\right) = 0.09 \cdot m^3$$

$$V_{crmsup} := l_{crmsup} \cdot \left(t_{crmsup} \cdot h_{crmsup} + 2 \cdot t_{crmsupfl} \cdot b_{crmsupfl}\right) = 0.124 \cdot m^3$$

$$V_{gird} := \left(h_{web} \cdot t_{web} + b_{fll} \cdot t_{fl} + b_{flu} \cdot t_{fl}\right) \cdot n_{gird} \cdot l_{span} \cdot n_{span} = 14.08 \cdot m^3$$

$$V_{cross} := n_{crfield} \cdot V_{crfield} + n_{cresup} \cdot V_{cresup} + n_{crmsup} \cdot V_{crmsup} = 1.106 \cdot m^3$$

$$m_{steel} := \rho_{steel} \cdot \left(V_{gird} + V_{cross}\right) = 118.453 \cdot tonne$$

$$m_2 := \frac{m_{steel}}{l_{span} \cdot n_{span}} = 2.692 \cdot \frac{tonne}{m}$$

Exposed steel area

In total:

$$A_{steelexp} := n_{gird} \cdot l_{span} \cdot n_{span} \cdot \left(h_{web} \cdot 2 + b_{fll} \cdot 2 + b_{flu}\right) \ldots \\ + n_{crfield} \cdot l_{crfield} \cdot \left(h_{crfield} \cdot 2 + b_{crfieldfl} \cdot 4\right) \ldots \\ + n_{cresup} \cdot l_{cresup} \cdot \left(h_{cresup} \cdot 2 + b_{cresupfl} \cdot 4\right) \ldots \\ + n_{crmsup} \cdot l_{crmsup} \cdot \left(h_{crmsup} \cdot 2 + b_{crmsupfl} \cdot 4\right) \quad = 918.505\, m^2$$

Exposed area for main beams:

$$n_{gird} \cdot l_{span} \cdot n_{span} \cdot \left(h_{web} \cdot 2 + b_{fll} \cdot 2 + b_{flu}\right) = 756.8\, m^2$$

Exposed area for cross beams:

$$n_{crfield} \cdot l_{crfield} \cdot \left(h_{crfield} \cdot 2 + b_{crfieldfl} \cdot 4\right) \ldots \\ + n_{cresup} \cdot l_{cresup} \cdot \left(h_{cresup} \cdot 2 + b_{cresupfl} \cdot 4\right) \ldots \\ + n_{crmsup} \cdot l_{crmsup} \cdot \left(h_{crmsup} \cdot 2 + b_{crmsupfl} \cdot 4\right) \quad = 161.705\, m^2$$

Amount of surfacing (asphalt)

$$t_{HABS} := 45mm$$

$$t_{PGJA} := 30mm$$

$$t_{iso} := 5mm$$

$$t_{MAB} := 40mm$$

$$t_{AG} := 120mm$$

$$A_{kb} := l_{tot} \cdot \left(b_{kf} \cdot 2 + b_{mid}\right) = 981\, m^2$$

$$A_{gc} := l_{tot} \cdot b_{gc} = 109\, m^2$$

$$m_{asp} := \rho_{asp} \cdot \left[\left(t_{HABS} + t_{PGJA}\right) \cdot A_{kb} + \left(t_{MAB} + t_{AG} + t_{PGJA}\right) \cdot A_{gc}\right] = 224.398 \cdot tonne$$

Amount of reinforcement

$m_{reinf} := 36tonne$ assumed, based on 150 kg/m^3

Appendix D

Design of FRP Alternative 1
7 longitudinal beams with transversal FRP deck

Bridge dimensions

$B := 20m$ Total width of the bridge

$L := 22m$ Span length

$n_{span} := 2$

$C_c := 2.8m$ Distance between girders

$o_{left} := 1.2m$ Overhang

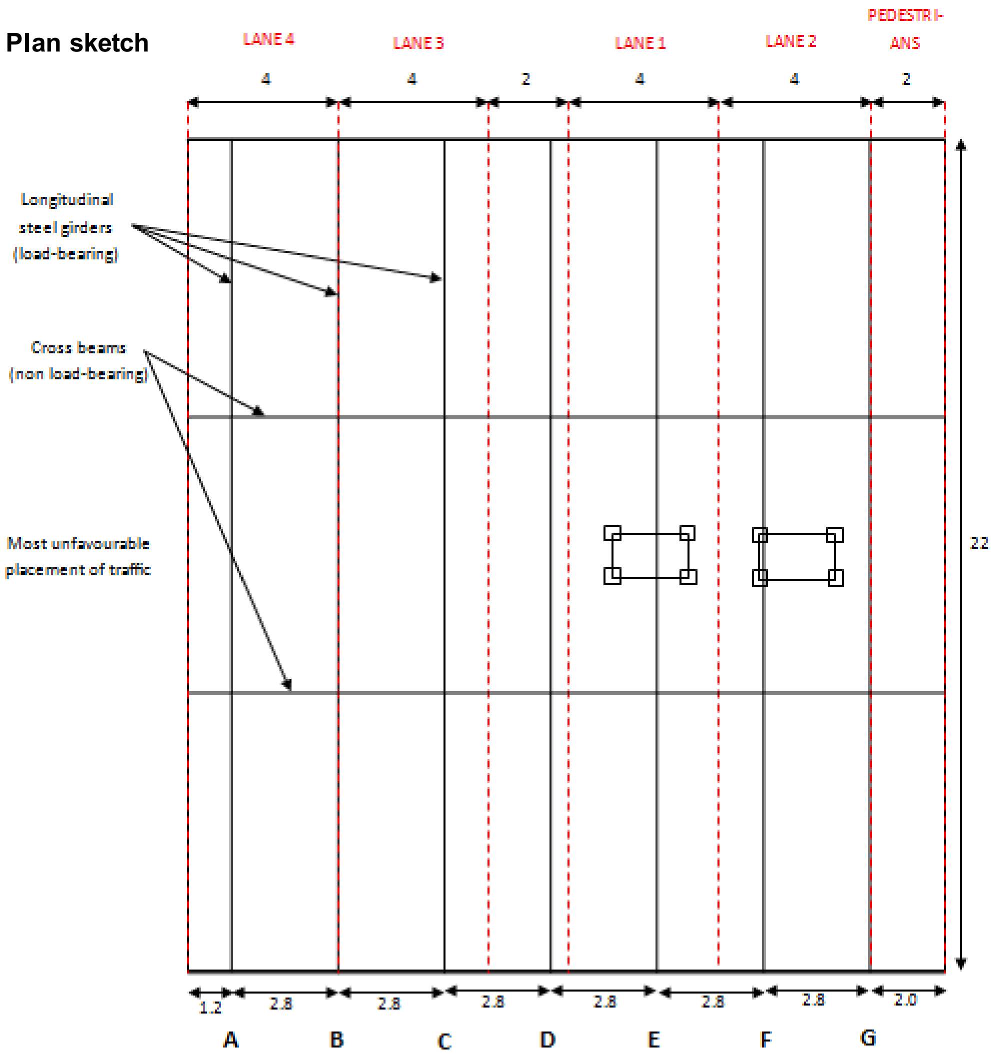
$o_{right} := 2m$

$\delta_{max} := \frac{C_c}{300} = 9.333 \times 10^{-3} m$ Maximum allowed deflection for the FRP deck

$b_{gc} := 2m$ width of pedestrian/bike lane

$b_{kf} := 4m$ width of lane

Plan sketch



FRP material properties

$E_{xx} := 20 \frac{kN}{mm^2}$ elasticity modulus in x-direction

$E_{yy} := 18 \frac{kN}{mm^2}$ elasticity modulus in y-direction

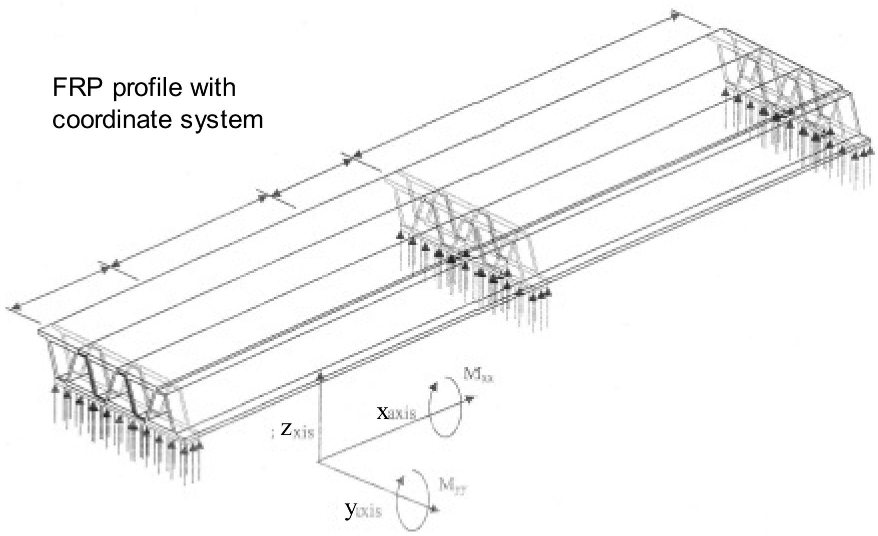
$I_{xx} := 342600000 \frac{mm^4}{m}$ 2nd moment of inertia around the x-axis

$I_{yy} := 409800000 \frac{mm^4}{m}$ 2nd moment of inertia around the y-axis

$$EI_{xx} := E_{yy} \cdot I_{xx} = 6.167 \times 10^3 \frac{1}{m} \cdot kN \cdot m^2$$

$$EI_{yy} := E_{xx} \cdot I_{yy} = 8.196 \times 10^6 \frac{1}{m} \cdot N \cdot m^2$$

FRP profile with coordinate system



Steel properties

Steel quality S355

$f_y := 355MPa$ $E_{st} := 210GPa$

Loads

Selfweight

In SLS:

$$g_{FRP} := 103.69 \cdot g \frac{kg}{m^2} = 1.017 \cdot \frac{kN}{m^2}$$ weight FRP

$$\rho_{steel} := 7800 \frac{kg}{m^3}$$

$$g_{steel} := \rho_{steel} g = 76.492 \cdot \frac{kN}{m^3}$$ weight steel

$$g_{asp} := 23 \frac{kN}{m^3}$$ weight asphalt

$$g_{as} := g_{asp} \cdot 0.1m = 2.3 \cdot \frac{kN}{m^2}$$ assumed asphalt layer 1 dm

$$g_{par} := 2 \frac{kN}{m}$$ weight parapet

In ULS:

$$g_{FRPuls} := g_{FRP} \cdot 1.35 = 1.373 \cdot \frac{kN}{m^2}$$

$$g_{aspuls} := g_{as} \cdot 1.35 = 3.105 \cdot \frac{kN}{m^2}$$

$$g_{paruls} := g_{par} \cdot 1.35 = 2.7 m \cdot \frac{kN}{m^2}$$

$$g_{steeluls} := g_{steel} \cdot 1.35 = 103.264 \frac{kN}{m^3}$$

Traffic loads according to LM1 in Eurocode and additions from Trafikverket

uniformly distributed loads for lane 1-4 and remaining areas, the pedestrian lane was regarded as a remaining area

$$q_{l1} := 9 \frac{kN}{m}$$

$$q_{l2} := 2.5 \frac{kN}{m}$$

$$q_{l3} := q_{l2} = 2.5 \cdot \frac{kN}{m}$$

$$q_{l4} := q_{l2} = 2.5 \cdot \frac{kN}{m}$$

$$q_{re} := q_{l2} = 2.5 \cdot \frac{kN}{m}$$

$Q_1 := 150kN$ point loads, half axle load (i.e. one wheel) for lane 1-3

$Q_2 := 100 \cdot kN$

$Q_3 := 50kN$

The traffic loads can be reduced according to Swedish National annex to Eurocode from Trafikverket)

$\alpha_{q1} := 0.7$ reduction factors for uniformly distributed loads

$\alpha_{qi} := 1$

$\alpha_{qr} := 1$

reduction factors for point loads

$\alpha_{Q1} := 0.9$

$\alpha_{Q2} := \alpha_{Q1}$

$\alpha_{Q3} := 0$

$q_{l1red} := q_{l1} \cdot \alpha_{q1} = 6.3 \cdot \frac{kN}{m}$ reduced uniformly distributed loads

$q_{lired} := q_{l2} \cdot \alpha_{qi} = 2.5 \cdot \frac{kN}{m}$

$q_{rered} := q_{re} \cdot \alpha_{qr} = 2.5 \cdot \frac{kN}{m}$

$Q_{1red} := Q_1 \cdot \alpha_{Q1} = 1.35 \times 10^5 N$ reduced point loads

$Q_{2red} := Q_2 \cdot \alpha_{Q2} = 90 \cdot kN$

$Q_{3red} := Q_3 \cdot \alpha_{Q3} = 0 \cdot kN$ All traffic loads above are applicable in SLS.

Loads in ULS

$q_{l1ULS} := q_{l1red} \cdot 1.5 = 9.45 \cdot \frac{kN}{m}$

$q_{l2ULS} := q_{lired} \cdot 1.5 = 3.75 \cdot \frac{kN}{m}$

$q_{l3ULS} := q_{lired} \cdot 1.5 = 3.75 \cdot \frac{kN}{m}$

$q_{l4ULS} := q_{lired} \cdot 1.5 = 3.75 \cdot \frac{kN}{m}$

$q_{reULS} := q_{rered} \cdot 1.5 = 3.75 \cdot \frac{kN}{m}$

$$Q_{1ULS} := Q_{1red} \cdot 1.5 = 202.5 \cdot kN$$

$$Q_{2ULS} := Q_{2red} \cdot 1.5 = 135 \cdot kN$$

$$Q_{3ULS} := Q_{3red} \cdot 1.5 = 0$$

Application of loads

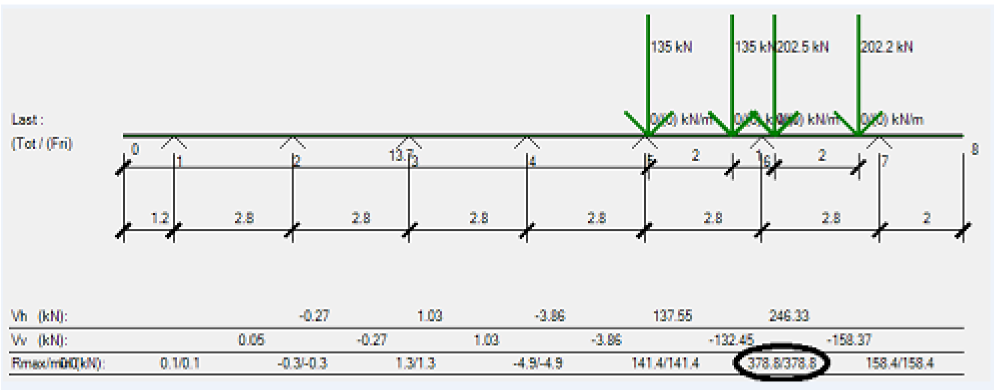
These loads were applied to a model made in the software SBBalk. A section of the bridge was modelled as a beam and then different scenarios were tried until the worst combination of lanes was found.
Uniformly distributed loads and point loads were modelled separately.
This resulted in support F (i.e. the fifth beam) getting the highest reaction force.

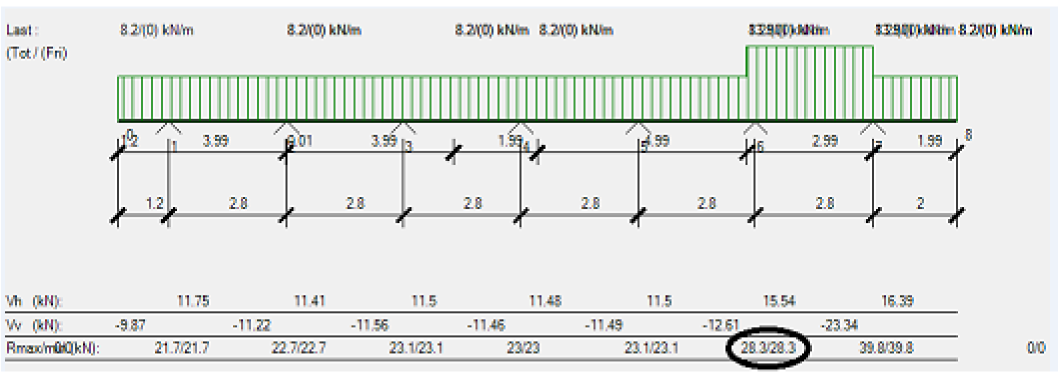
A new model of the length of the bridge was made. The reaction force from the uniformly distributed loads was applied as a distributed load over the length of the beam and the reaction force from the point loads (the tandem system) was applied in two points (one for each axle).
The point load was applied in the middle of the span when moment and deflection was evaluated and near the support when shear force was evaluated.

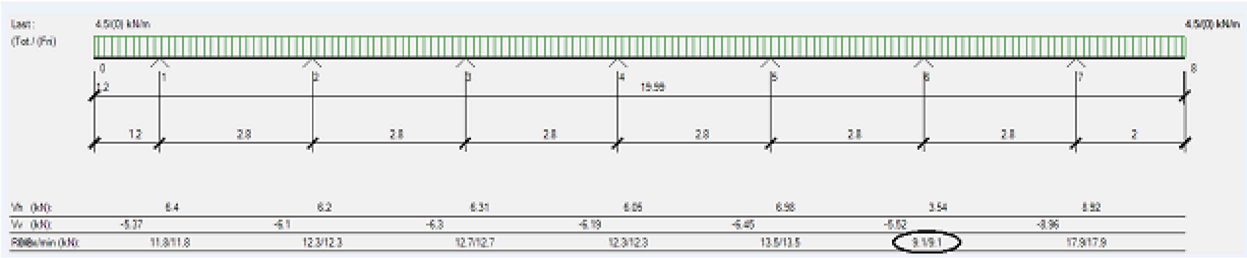
$q_F := 28.3 \frac{kN}{m}$ $\qquad g_F := 9.1 \frac{kN}{m}$ $\qquad$ see figures below

$P_F := 378.8kN$

Thus, we get a uniformly distributed load of R_F kN/m along the beam, named q_F and two point loads P_F from the axles.







Check moment, deflection and shear force

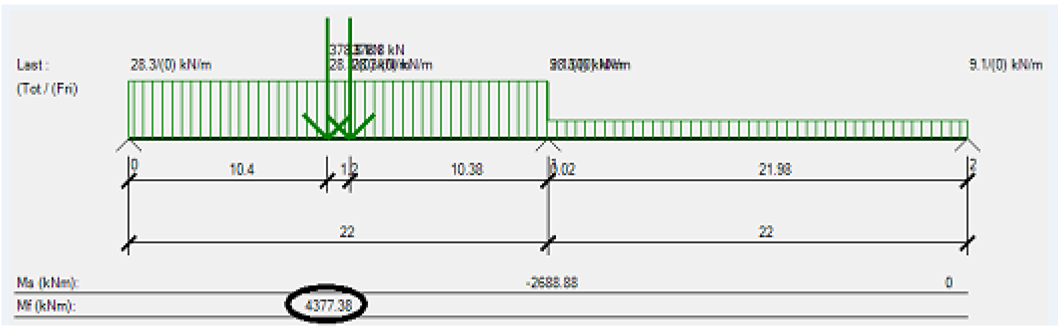
$$M_{Rd} := \frac{f_y \cdot I_{st}}{z} \qquad V_{Rd} := \frac{f_y \cdot I_{st} \cdot t_{web}}{S_{steel}}$$

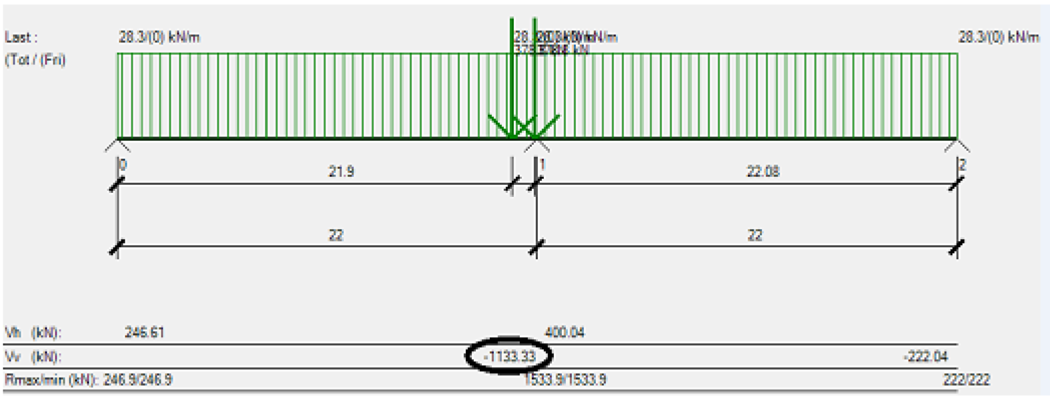
Design moment and shear force from SBBalk:
maximum moment is taken with two pointloads, c-c1.2 m centered around midspan. Maximum shear force is taken with the loads just next to the support. Values can be found in sbbalk-files: spoa22.sbb and c-c2800mm no dist load.sbb and c-c2800mm no point loads.sbb
The values are calculated using ULS values.

$M_{sed} := -3.70938 MN \cdot m$ see figures below

$M_{fed} := 4377.38 kN \cdot m$

$V_{sed} := 1133.33 kN$





Create an I-section that fulfills the demands for moment and shear force:

$$h_{st} := 1200mm$$

$$z := \frac{h_{st}}{2} = 0.6\,m$$

$$t_{web} := 19mm$$

$$t_{fl} := 35mm$$

$$b_{fl} := 350mm$$

$$h_{web} := h_{st} - 2 \cdot t_{fl} = 1.13\,m$$

$$S_{steel} := t_{fl} \cdot b_{fl} \cdot \left(\frac{h_{st}}{2} - \frac{t_{fl}}{2}\right) + t_{web} \cdot \frac{\left(\frac{h_{st}}{2} - t_{fl}\right)^2}{2} = 0.01 \cdot m^3$$

$$I_{st} := \frac{h_{web}^{\,3} \cdot t_{web}}{12} + 2 \cdot \frac{b_{fl} \cdot t_{fl}^{\,3}}{12} + 2b_{fl} \cdot t_{fl} \cdot \left(\frac{h_{st}}{2} - \frac{t_{fl}}{2}\right)^2 = 1.06 \times 10^{10} \cdot mm^4$$

$$M_{Rd} := \frac{f_y \cdot I_{st}}{z} = 6.272 \times 10^3 \cdot kN \cdot m \qquad V_{Rd} := \frac{f_y \cdot I_{st} \cdot t_{web}}{S_{steel}} = 7.031 \times 10^3 \cdot kN$$

$$M_{Rd} > \left|M_{fed}\right| = 1 \qquad V_{Rd} > V_{sed} = 1$$

Utilization rate for moment and shear force:

$$\frac{\left|M_{fed}\right|}{M_{Rd}} = 0.698 \qquad \frac{V_{sed}}{V_{Rd}} = 0.161$$

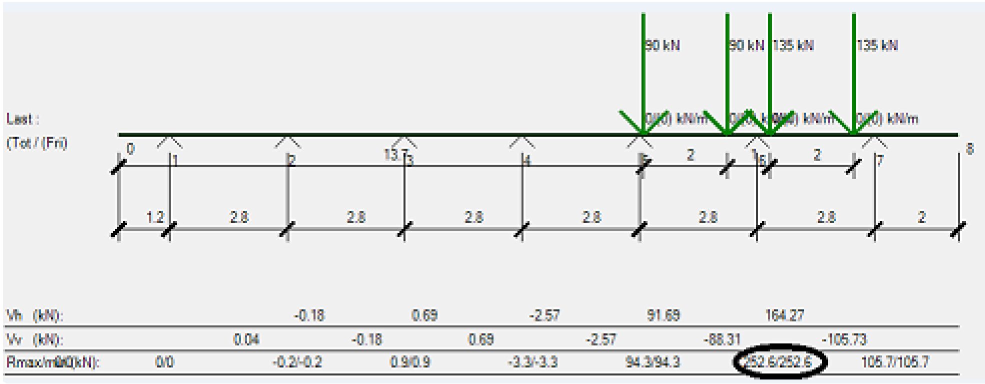
It works! :)

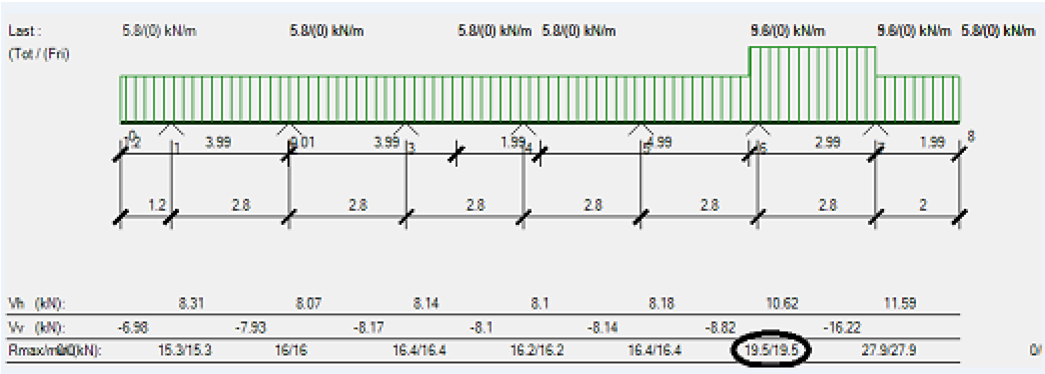
$$E_{st} \cdot I_{st} = 2.226 \times 10^9 \cdot N \cdot m^2$$

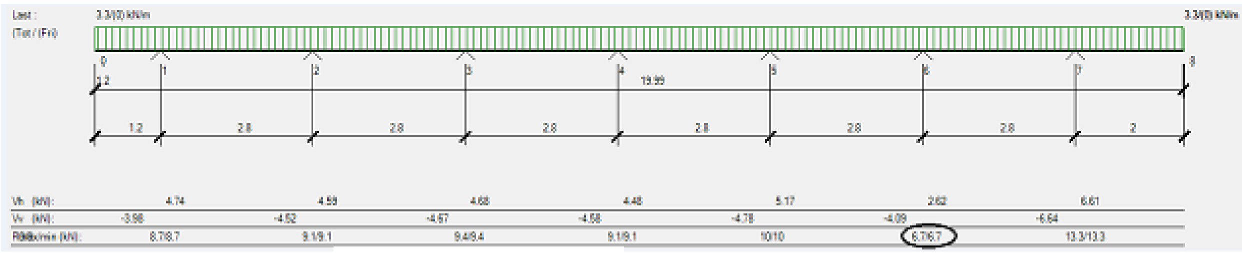
SBBalk shows which flexural rigidity (EI) is needed to get a deflection within limits. The I-section is adapted until it gets an EI large enough. This time, SLS values found in SBBalk are used.

$$P_{FSLS} := 252.6kN \qquad g_{FSLS} := 6.7\frac{kN}{m}$$

$$q_{FSLS} := 19.5kN$$







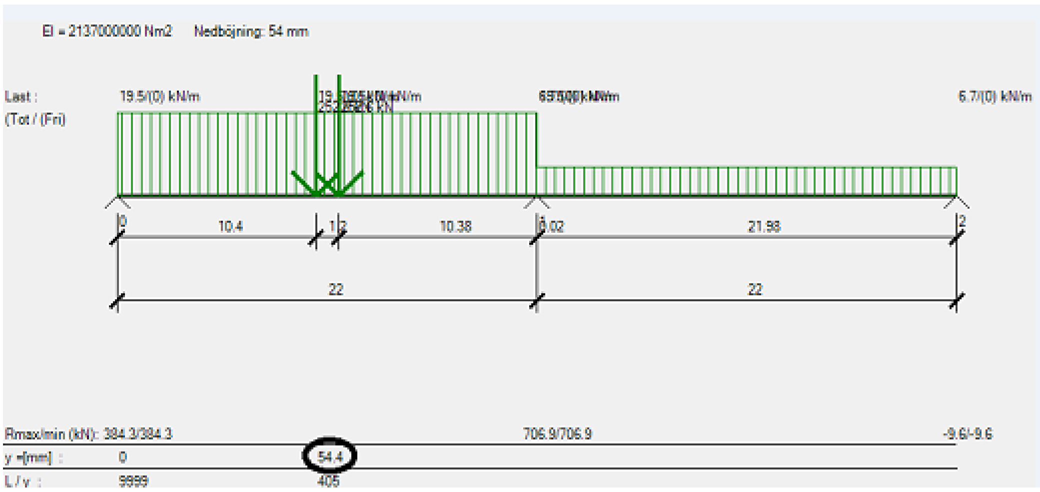
Limiting deflection is: $\delta_{max2} := \frac{L}{400} = 0.055\,m$

To achieve this we need an EI>2.226e9 Nm^2 according to SBBalk. Thus we need:

$$EI_b := 2.226\cdot 10^{9} N\cdot m^{2}$$

$$I_b := \frac{EI_b}{E_{st}} = 1.06\times 10^{10}\cdot mm^{4}$$ needed value of I

$I_{st} > I_b = 1$ $\qquad$ $\frac{I_b}{I_{st}} = 1$



Weight of beams:

$n_{beam} := 7$ number of longitudinal beams

$A_{st} := h_{web} \cdot t_{web} + 2t_{fl} \cdot b_{fl} = 4.597 \times 10^{4} \cdot mm^{2}$ area of the cross-section of the I-beam

$V_{st} := A_{st} \cdot L \cdot n_{span} = 2.023 \cdot m^{3}$ volume of one beam

$A_{cross} := 0.018 \cdot m^{2}$ crossings are assumed to be the same as in the steel/concrete composite bridge

$l_{cross} := C_c = 2.8\,m$

$P_{cross} := A_{cross} \cdot \frac{l_{cross}}{2} \cdot \rho_{steel} \cdot g = 1.928 \cdot kN$ the weight/force form the cross beams has not been included. As can be seen, the load from them is negligible

$n_{cross} := 28$ number of cross beams

$m_{st} := V_{st} \cdot \rho_{steel} \cdot n_{beam} + A_{cross} \cdot l_{cross} \cdot n_{cross} \cdot \rho_{steel} = 121.446 \cdot tonne$ total weight of steel beams and crossings

$m_2 := \frac{m_{st}}{L \cdot n_{span}} = 2.76 \cdot \frac{tonne}{m}$ steel weight/meter

Exposed steel area

$A_{explong} := n_{beam} \cdot L \cdot n_{span} \cdot \left(2 \cdot h_{web} + 3 \cdot b_{fl}\right) = 1.019 \times 10^{3}\,m^{2}$

$h_{webcross} := 778.6mm$

$b_{flcross} := 257mm$

$A_{expcross} := n_{cross} \cdot l_{cross} \cdot \left(2 \cdot h_{webcross} + 4 \cdot b_{flcross}\right) = 202.68\,m^{2}$

$A_{paint1} := A_{explong} + A_{expcross} = 1.222 \times 10^{3}\,m^{2}$ total exposed steel area

Appendix E

Design of FRP Alternative 2
4 longitudinal beams , load-bearing cross beams

Values from calculation of alternative 1 and existing bridge, for comparison:

$m_{bet} := 2.692 \frac{tonne}{m}$ weight of the beams of the steel/concrete composite bridge.

$m_{alt1} := 2.636 \frac{tonne}{m}$ weight of the steel beams in alternative 1, transversal FRP-deck

Alternativ 2, longitudinal FRP deck supported on crossbeams and longitudinal beams

The FRP carries load in both directions and works as a plate, this way we can handle the local deflection in the FRP.
When designing the beams tough, it is assumed that the longitudinal beams carries the transversal beams which then carries the FRP (i.e no plate action but on the safe side).

Bridge dimensions

$B := 20m$ Total width of the bridge

$L_{span} := 22m$ Span length

$C_{lb} := 4.67m$ Distance between longitudinal beams

$C_{cb} := 3.67m$ distance between cross beams

$o_{can} := 3m$ Overhang of the cantilever

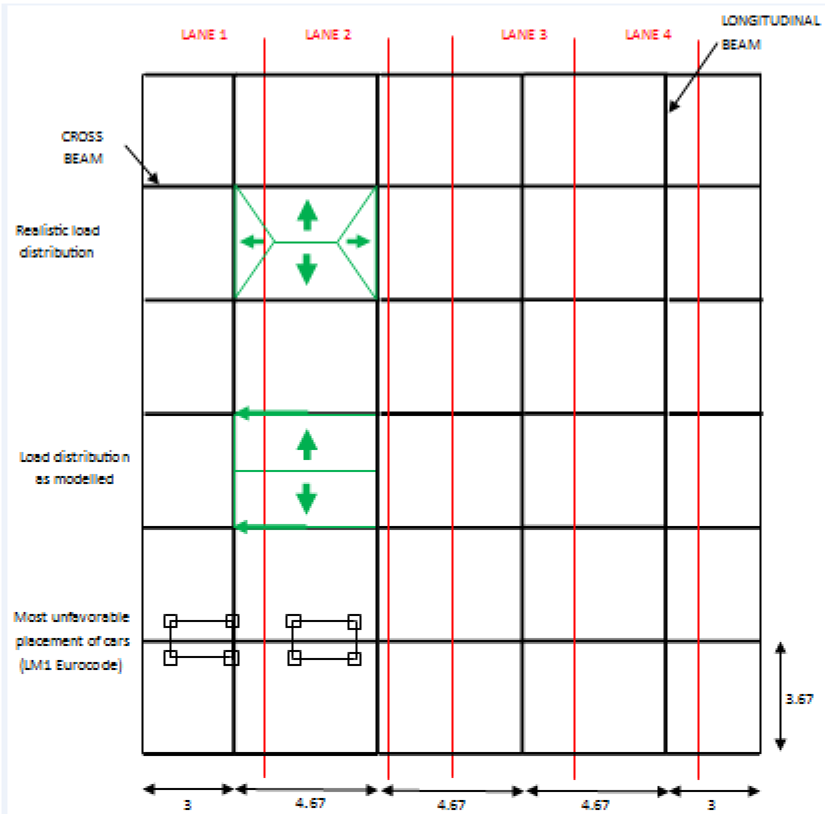
$n_{span} := 2$

$\delta_{max} := \frac{C_{cb}}{300} = 0.012\,m$ Maximum allowed deflection for the FRP deck

$b_{gc} := 2m$ width of pedestrian/bike lane

$b_{kf} := 4m$ width of lane

Plan sketch of the bridge
Green areas and arrows shows the model.



FRP material properties

$E_{xx} := 20 \frac{kN}{mm^2}$ elasticity modulus in x-direction of the material

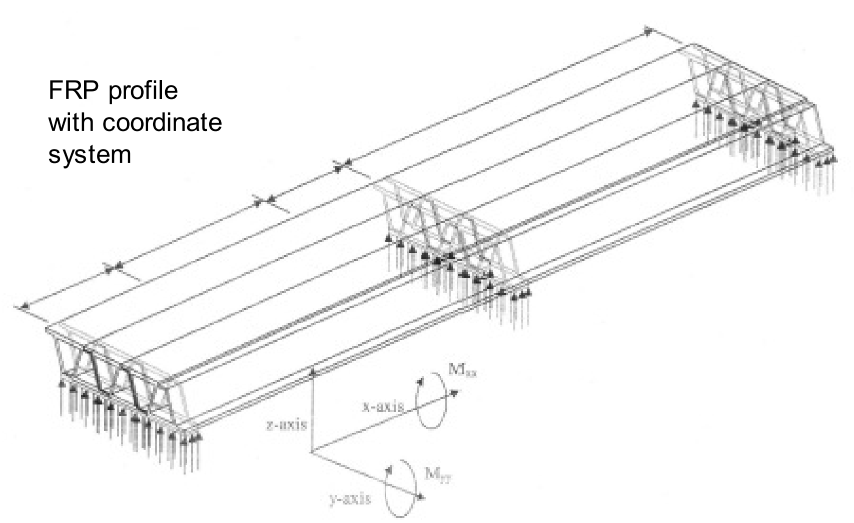
$E_{yy} := 18 \frac{kN}{mm^2}$ elasticity modulus in y-direction

$I_{xx} := 342600000 \frac{mm^4}{m}$ second moment of inertia around the x-axis

$I_{yy} := 409800000 \frac{mm^4}{m}$ second moment of inertia around the y-axis

$$EI_{xx} := E_{yy} \cdot I_{xx} = 6.167 \times 10^3 \frac{1}{m} \cdot kN \cdot m^2$$

$$EI_{yy} := E_{xx} \cdot I_{yy} = 8.196 \times 10^3 \frac{1}{m} \cdot kN \cdot m^2$$



Steel properties

$f_y := 355 \cdot 10^6 Pa$

$E := 210GPa$

Loads

Self-weight

$g_{FRP} := 1.017 \frac{kN}{m^2}$

$\rho_{steel} := 7800 \frac{kg}{m^3}$

$g_{asp} := 2.3 \frac{kN}{m^2}$

$g_{par} := 2 \frac{kN}{m}$

$g_{FRPuls} := g_{FRP} \cdot 1.35 = 1.373 \cdot \frac{kN}{m^2}$

$g_{aspuls} := g_{asp} \cdot 1.35 = 3.105 \cdot \frac{kN}{m^2}$

$g_{paruls} := g_{par} \cdot 1.35 = 2.7 m \cdot \frac{kN}{m^2}$

Traffic load according to Eurocode, LM1 with additions from Trafikverket

$b_{trib} := 3.67m$ — width contributing to the loading of one cross-beam

$q_{l1} := 9\frac{kN}{m^2} \cdot b_{trib} = 33.03 \cdot \frac{kN}{m}$ — uniformly distributed loads for lane 1-4 and remaining areas

$q_{l2} := 2.5\frac{kN}{m^2} \cdot b_{trib} = 9.175 \cdot \frac{kN}{m}$

$q_{l3} := 2.5\frac{kN}{m^2} \cdot b_{trib} = 9.175 \cdot \frac{kN}{m}$

$q_{l4} := 2.5\frac{kN}{m^2} \cdot b_{trib} = 9.175 \cdot \frac{kN}{m}$

$q_{re} := q_{l4}$

$Q_1 := 150kN$ — points loads, half axle load (i.e. one wheel) for lane 1-3

$Q_2 := 100 \cdot kN$

$Q_3 := 50kN$

The traffic loads can be reduced according to Swedish National annex to Eurocode (form Trafikverket)

$\alpha_{q1} := 0.7$ — reduction factors for uniformly distributed loads

$\alpha_{qi} := 1$

$\alpha_{qr} := 1$

$\alpha_{Q1} := 0.9$ — reduction factors for point loads

$\alpha_{Q2} := \alpha_{Q1}$

$\alpha_{Q3} := 0$

$q_{l1red} := q_{l1} \cdot \alpha_{q1} = 23.121 \cdot \frac{kN}{m}$ — reduced uniformly distributed loads

$q_{lired} := q_{l2} \cdot \alpha_{qi} = 9.175 \cdot \frac{kN}{m}$

$q_{rered} := q_{re} \cdot \alpha_{qr} = 9.175 \cdot \frac{kN}{m}$

$Q_{1red} := Q_1 \cdot \alpha_{Q1} = 1.35 \times 10^5 \, N$ reduced point loads

$Q_{2red} := Q_2 \cdot \alpha_{Q2} = 90 \cdot kN$

$Q_{3red} := Q_3 \cdot \alpha_{Q3} = 0 \cdot kN$ All loads above are valid for SLS

Loads in ULS

$q_{l1ULS} := q_{l1red} \cdot 1.5 = 34.681 \cdot \frac{kN}{m}$ uniformly distributed loads

$q_{l2ULS} := q_{lired} \cdot 1.5 = 13.762 \cdot \frac{kN}{m}$

$q_{l3ULS} := q_{lired} \cdot 1.5 = 13.762 \cdot \frac{kN}{m}$

$q_{l4ULS} := q_{lired} \cdot 1.5 = 13.762 \cdot \frac{kN}{m}$

$q_{reULS} := q_{rered} \cdot 1.5 = 13.762 \cdot \frac{kN}{m}$

$Q_{1ULS} := Q_{1red} \cdot 1.5 = 202.5 \cdot kN$ points loads, half axle load for lane 1-3

$Q_{2ULS} := Q_{2red} \cdot 1.5 = 135 \cdot kN$

$Q_{3ULS} := Q_{3red} \cdot 1.5 = 0$

DESIGN OF CROSS BEAMS

Load application on cross-beams, interior span (not cantilevers)

Each cross-beam is assumed to have a tributary width of 3.67m (same as the distance between the beams). Point loads were applied for the axle loads, car assumed to be placed in the middle of "lane 1".

$b_{trib} := 3.67m$ $l_{cross} := 4.67m$ $l_{can} := 3m$

$w_{FRPuls} := g_{FRPuls} \cdot b_{trib} = 5.039 \cdot \frac{kN}{m}$ $w_{FRPsls} := g_{FRP} \cdot b_{trib} = 3.732 \cdot \frac{kN}{m}$

$w_{aspuls} := g_{aspuls} \cdot b_{trib} = 11.395 \cdot \frac{kN}{m}$ $w_{asp} := g_{asp} \cdot b_{trib} = 8.441 \cdot \frac{kN}{m}$

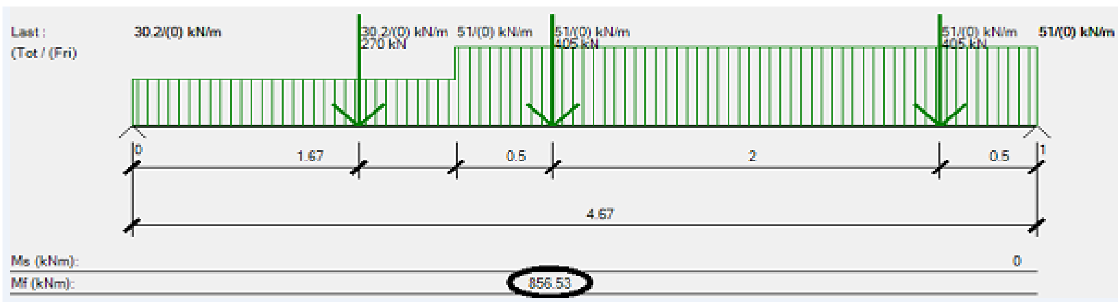
Check moment, deflection and shear force

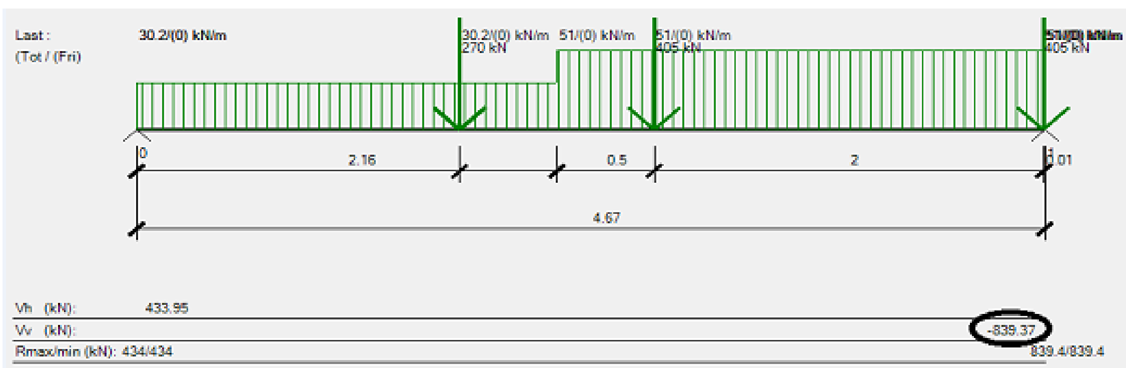
$$M_{Rd} := \frac{f_y \cdot I_{st}}{z} \qquad V_{Rd} := \frac{f_y \cdot I_{st} \cdot t_{web}}{S_{steel}} \qquad f_{yIPE} := 275\text{MPa}$$

Design moment and shear force were found in SBBalk (file crossbeam ULS.sbb).

$$M_{Ed} := 856.5\text{kN} \cdot \text{m}$$

$$V_{Ed} := 839\text{kN}$$





A standard section that fulfills the demands on moment and shear force was used. Here, IPE600. Properties according to data sheet.

$$I_{st} := 92080 \cdot 10^4 \cdot \text{mm}^4$$

$$h := 600\text{mm}$$

$$z := \frac{h}{2} = 0.3\,\text{m}$$

$$b_{fl} := 220\text{mm}$$

$$t_{fl} := 19\text{mm}$$

$$t_{web} := 12\text{mm}$$

$$h_{web} := h - 2 \cdot t_{fl} = 0.562\,m$$

$$S_{steel} := t_{fl} \cdot b_{fl} \cdot \left(\frac{h}{2} - \frac{t_{fl}}{2}\right) + \frac{t_{web} \cdot \left(\frac{h}{2} - t_{fl}\right)^2}{2} = 1.688 \times 10^{-3} \cdot m^3$$

$$M_{Rd} := \frac{f_y \cdot I_{st}}{z} = 1.09 \times 10^3 \cdot kN \cdot m \qquad V_{Rd} := \frac{f_y \cdot I_{st} \cdot t_{web}}{S_{steel}} = 2.324 \times 10^3 \cdot kN$$

$$M_{Rd} > |M_{Ed}| = 1 \qquad V_{Rd} > V_{Ed} = 1$$

Utilization rate for moment and shear force:

$$\frac{M_{Ed}}{M_{Rd}} = 0.786 \qquad \frac{V_{Ed}}{V_{Rd}} = 0.361$$

Works well.

$$EI := E \cdot I_{st} = 1.934 \times 10^8 \cdot N \cdot m^2$$

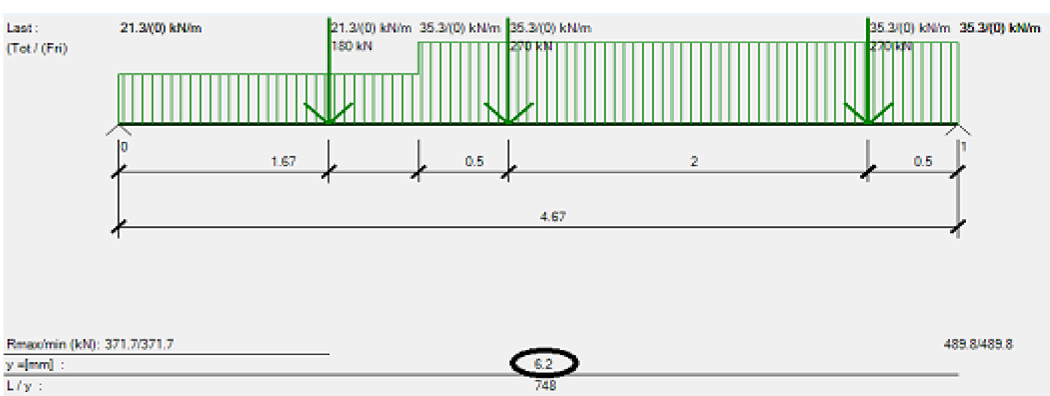
SBBalk shows what flexural rigidtiy (EI) is needed to get a deflection within limits. The I-section is adapted until it gets an EI large enough. This time, SLS values are applied. Used values can be found crossbeam SLS.sbb.

Limiting deflection is:

$$\delta_{lim} := \frac{l_{cross}}{400} = 0.012\,m \qquad \delta_{SBBalk} := 6.2mm$$

$$\frac{\delta_{SBBalk}}{\delta_{lim}} = 0.531$$

Choose IPE600 for the cross-beams, deflection in crossbeams is not considered (as in practice) but is, as can be seen, quite good.



Weight of cross-beams

$n_{trans} := 13$ — number of rows of cross-beams over the whole bridge

$m_{IPE600} := 122\,\frac{kg}{m}$ — weight/meter, IPE 600

$m_{cb} := l_{cross} \cdot m_{IPE600} = 0.57 \cdot tonne$ — weight of 1 cross-beam

$m_{canb} := l_{can} \cdot m_{IPE600} = 366\,kg$ — weight of 1 cantilever

The beam for the cantilever can be further optimized,
here the same is used as in the spans.

$n_{cross} := 3$ — number of interior spans with IPE-beams for each row of crossbeams

$n_{can} := 2$ — number of cantilevers in each row of crossbeams

$n_{cb} := n_{trans} \cdot n_{cross} = 39$

$m_{cbtot} := m_{cb} \cdot n_{cb} + m_{canb} \cdot n_{can} \cdot n_{cross} = 24.416 \cdot tonne$ — total weight of cross-beams

DESIGN OF LONGITUDINAL BEAMS

Application of loads on longitudinal beams

To find which beam is subjected to the highest load a number of combinations of different "lanes" according to Eurocode were tried out in SBBalk. It was found that the worst case was when the first lane was set as "lane 1" and the second lane is "lane 2". This gives the highest load on beam A (i.e. the first beam).
Weight of cross-beams is approx 5kN/m and disregarded in this first analysis.

It was found that when the tandem system is regarded we get a high point load on the affected longitudinal beam (P_{large}) and fron the cross-beams where only distributed load was considered we get a point load (P_{small}) on the longitudinal beam.
The cross-beams are regarded as simply supported between the longitudinal beams.
Values of the loads can be found in the files cb1-1 dist load ULS, cb2-2 dist load ULS, cb1-1 ULS.sbb and cb2-2 ULS.sbb.

$P_{large} := 963kN + 437kN = 1.4 \times 10^{3} \cdot kN$ — from tandem systems, LM1, found in SBBalk

$P_{small} := 153kN + 70.5kN = 223.5 \cdot kN$ — from distributed loads according to LM1.

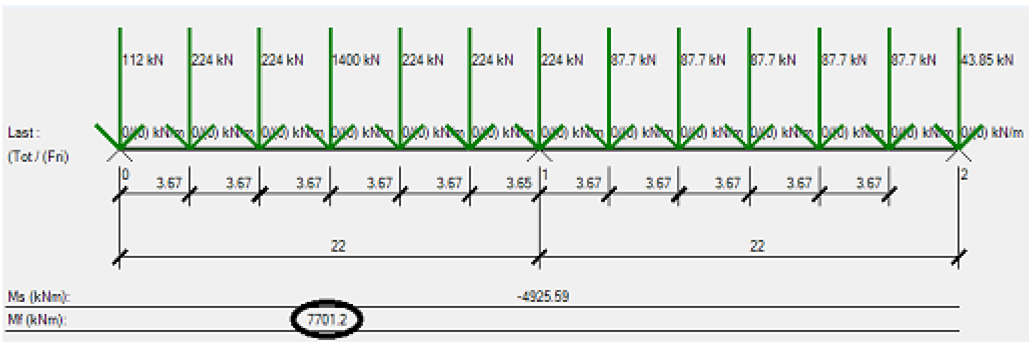
$P_{self} := 49.3kN + 38.4kN = 87.7 \cdot kN$ — self-weight of the slab only

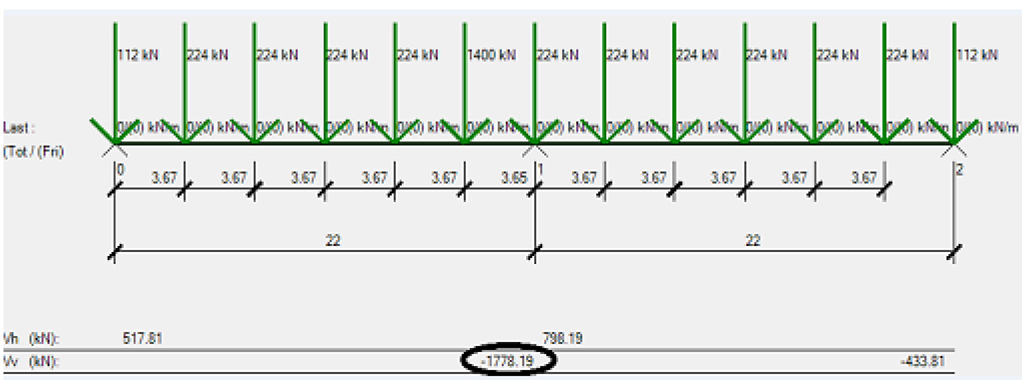
These loads were then applied to the entire bridge. To achieve the highest moment, the distributed load was applied to one of the spans as P_{small} and at the most centered cross-beam P_{large}, taking the tandem system into acocunt, was placed. To achieve the highest shear force P_{small} was applied in both spans and P_{large} was placed at the cross-beam closest to the mid-support. This can be found in the files: longitudinal ULS.sbb

$M_{fmax} := 7701.2kN \cdot m$ maximum field and support moment

$M_{smax} := -6018.83kN \cdot m$

$V_{max} := 1778.19kN$ maximum shear force





Check moment, shear force and deflection

$$M_{Rdl} := \frac{f_y \cdot I_{stl}}{z_l} \qquad V_{Rd} := \frac{f_y \cdot I_{stl} \cdot t_{webl}}{S_{steel}}$$

Create an I-section that fulfills the demands for moment and shear force.

$$h_l := 1100mm$$

$$z_l := \frac{h_l}{2} = 0.55\,m$$

$$b_{fl2} := 750mm$$

$$t_{fll} := 38mm$$

$$t_{webl} := 16.5mm$$

$$h_{webl} := h_l - 2 \cdot t_{fll} = 1.024\,m$$

$$I_{stl} := \frac{h_{webl}^{\ 3} \cdot t_{webl}}{12} + 2 \cdot \frac{b_{fl2} \cdot t_{fll}^{\ 3}}{12} + 2b_{fl2} \cdot t_{fll} \cdot \left(\frac{h_l}{2} - \frac{t_{fll}}{2}\right)^2 = 1.756 \times 10^{10} \cdot mm^4$$

$$S_{steel} := t_{fll} \cdot b_{fl2} \cdot \left(\frac{h_l}{2} - \frac{t_{fll}}{2}\right) + \frac{t_{webl} \cdot \left(\frac{h_l}{2} - t_{fll}\right)^2}{2} = 0.017 \cdot m^3$$

$$M_{Rdl} := \frac{f_y \cdot I_{stl}}{z_l} = 1.133 \times 10^4 \cdot kN \cdot m \qquad V_{Rdl} := \frac{f_y \cdot I_{stl} \cdot t_{webl}}{S_{steel}} = 5.945 \times 10^3 \cdot kN$$

$$M_{Rdl} > \left|M_{fmax}\right| = 1 \qquad V_{Rdl} > V_{max} = 1$$

Utilization rate for moment and shear force:

$$\frac{M_{fmax}}{M_{Rdl}} = 0.68 \qquad \frac{V_{max}}{V_{Rdl}} = 0.299$$

Good margin.

$$EI_l := E \cdot I_{stl} = 3.687 \times 10^9 \cdot N \cdot m^2$$

Use SBBalk to find which EI is needed to fulfill deflection demands. The EI needed is ~3.7e9 Nm^2. Loads are found according to the same principles as above, getting the values $P_{largesls}$ and $P_{smallsls}$ (found in longitudinal sls.sbb).

$P_{largesls} := 645.9kN + 294.4kN = 940.3 \cdot kN$

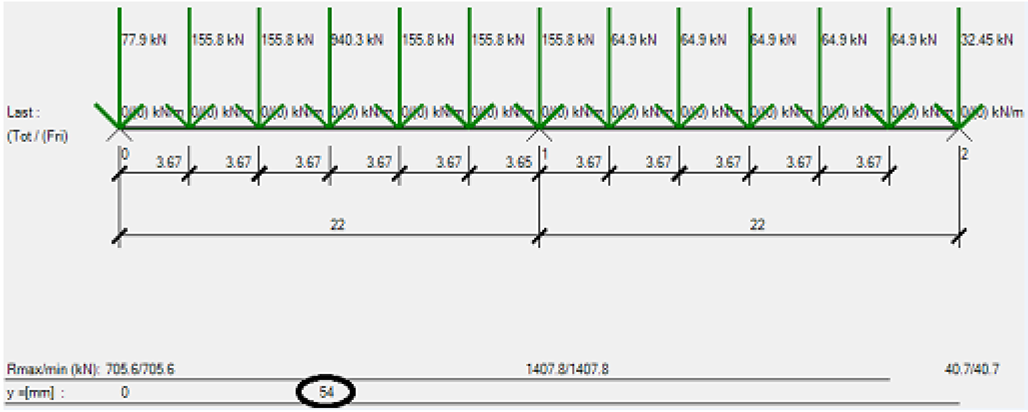
$P_{selfsls} := 36.5kN + 28.4kN = 64.9 \cdot kN$

$P_{smallsls} := 105.9kN + 49.9kN = 155.8 \cdot kN$

$$\delta_{liml} := \frac{L_{span}}{400} = 0.055 \cdot m$$

$\delta_l := 54mm$ deflection form SBBalk

$$\frac{\delta_l}{\delta_{liml}} = 0.982$$



Weight of longitudinal beams:

$m_{custom} := \left(h_{webl} \cdot t_{webl} + 2 \cdot b_{fl2} \cdot t_{fl1}\right) \cdot \rho_{steel} = 576.389 \frac{kg}{m}$ weight of the customized beam/meter

$n_{span} := 2$ number of spans

$n_{beam} := 4$ number of longitudinal beams

$m_{tot} := m_{custom} \cdot n_{beam} + \frac{m_{cbtot}}{n_{span} \cdot L_{span}} = 2.86 \cdot \frac{tonne}{m}$ total weight of steel (longitudinal and crossbeams) per meter

$m_1 := m_{tot} \cdot L_{span} \cdot n_{span} = 125.86 \cdot tonne$ total weight of steel in the bridge

The analysis above is made considering the FRPdeck on rigid supports (the beams). In reality the "supports" will be flexible and a load distribution could be accounted for.

Exposed steel area

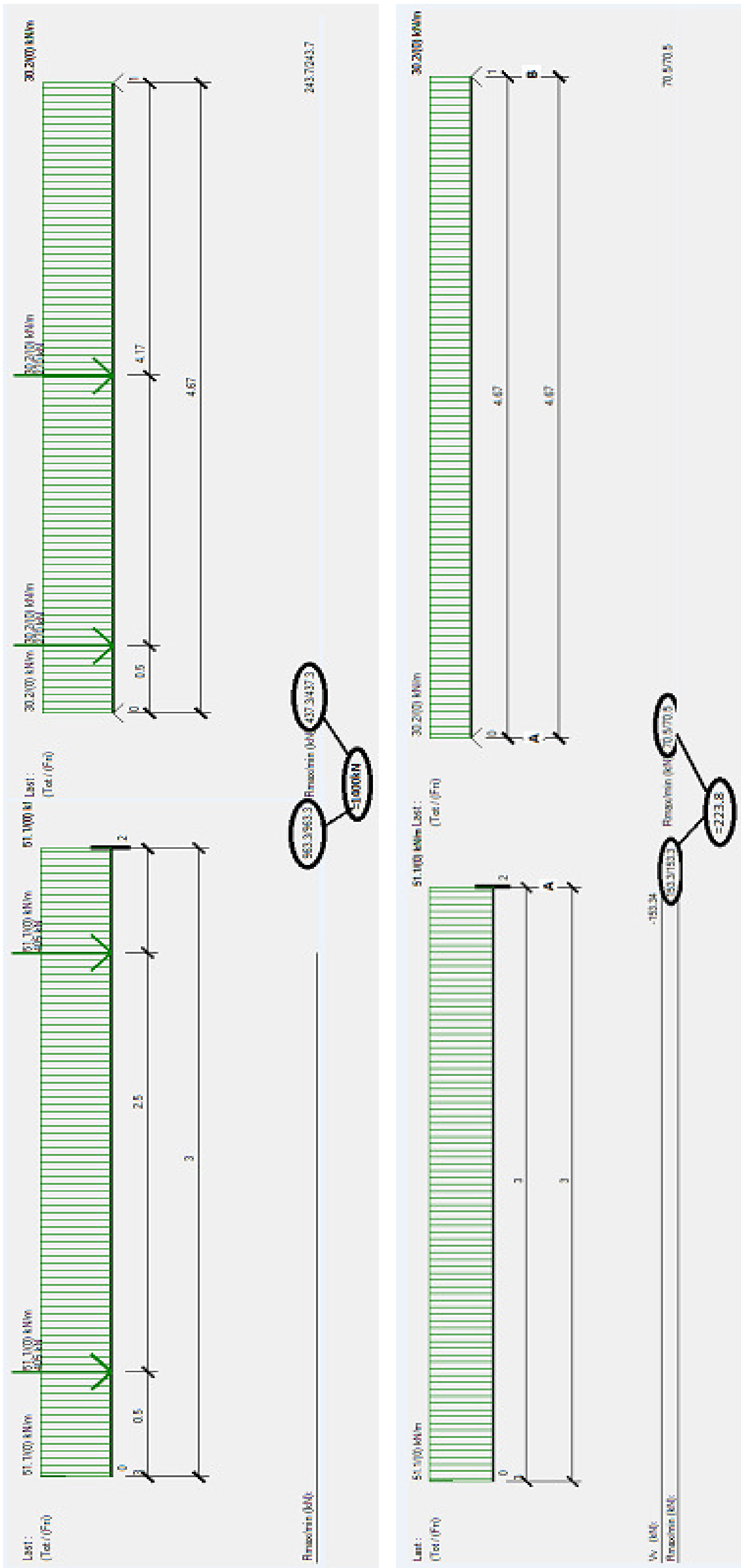
$$A_{explong} := n_{beam} \cdot L_{span} \cdot n_{span} \cdot \left(2 \cdot h_{webl} + 3 \cdot b_{fl2}\right) = 756.448\ m^2$$

$$h_{webcross} := h_{web} = 0.562\ m$$

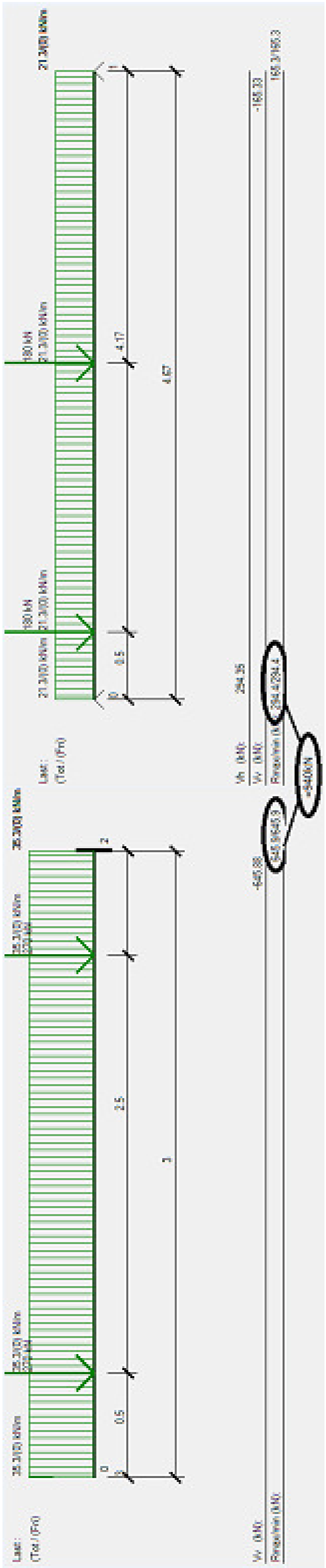
$$b_{flcross} := b_{fl} = 0.22\ m$$

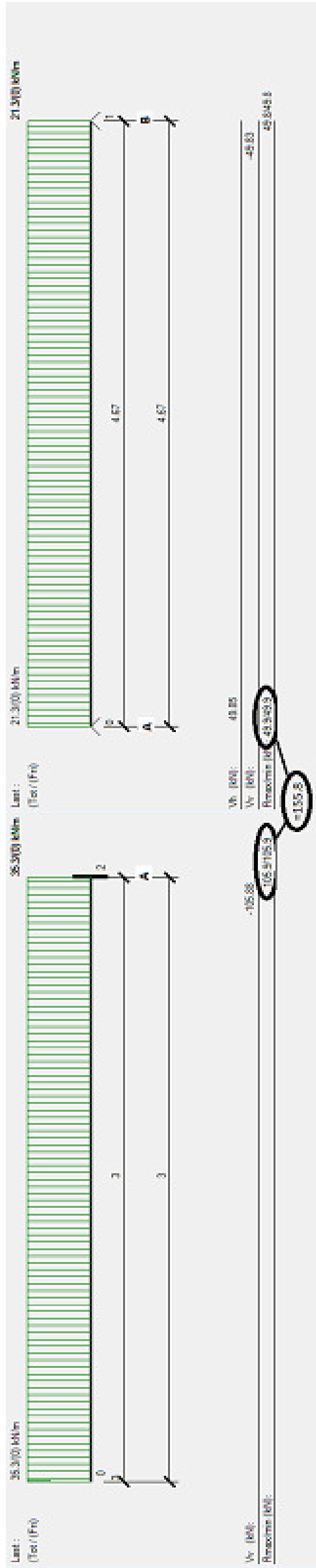
$$A_{expcross} := \left(n_{cb} \cdot l_{cross} + n_{can} \cdot n_{trans} \cdot l_{can}\right) \cdot \left(2 \cdot h_{webcross} + 3 \cdot b_{flcross}\right) = 464.072\ m^2$$

$$A_{paint2} := A_{explong} + A_{expcross} = 1.221 \times 10^3\ m^2$$ total exposed steel area



Point loads from cross-beams, ULS





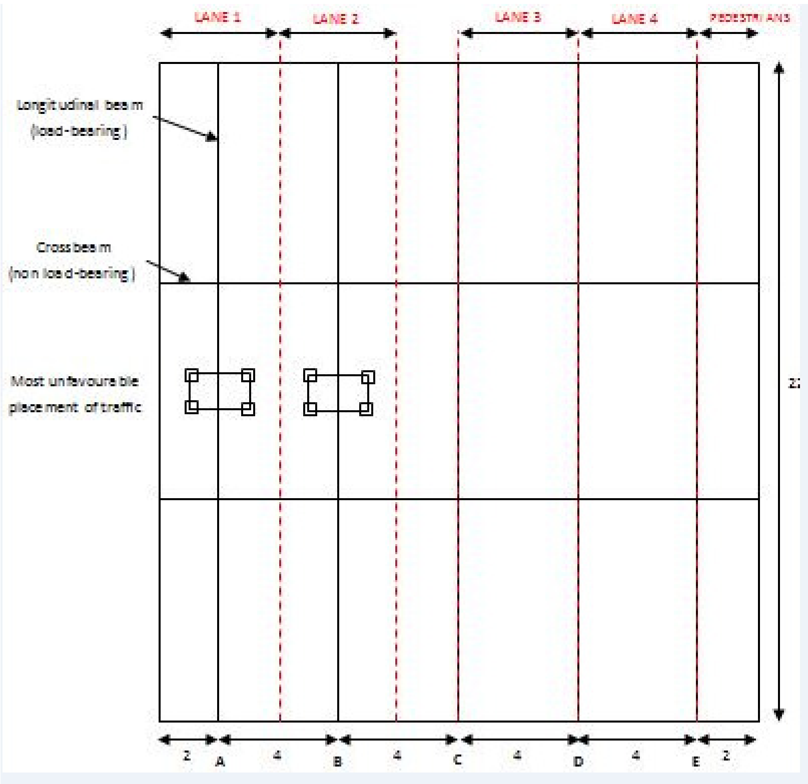
Point loads from cross-beams, SLS

Appendix F

Design of FRP Alternative 3
5 longitudinal beams with double transversal FRP deck

Bridge dimensions

$B := 20m$	Total width of the bridge
$L := 22m$	Span length
$C_c := 4m$	Distance between girders
$o_{left} := 2m$	Overhang
$o_{right} := 2m$	
$n_{span} := 2$	



$\delta_{max} := \dfrac{C_c}{300} = 0.013\,m$	Maximum allowed deflection for the FRP deck
$b_{gc} := 2m$	width of pedestrian/bike lane
$b_{kf} := 4m$	width of lane

FRP properties

$E_{xx} := 20\dfrac{kN}{mm^2}$ modulus of elasticity in x-direction of the material

$E_{yy} := 18\dfrac{kN}{mm^2}$ modulus of elasticity in y-direction of the material

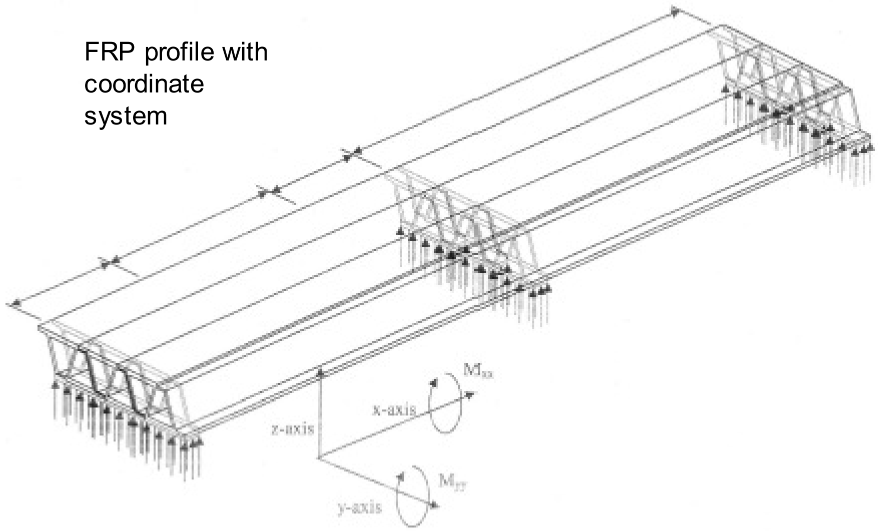
$I_{xx} := 342600000\dfrac{mm^4}{m}$ second moment of inertia around the x-axis

$I_{yy} := 409800000\dfrac{mm^4}{m}$ second moment of inertia around the y-axis

$$EI_{xx} := E_{yy} \cdot I_{xx} = 6.167 \times 10^3 \frac{1}{m} \cdot kN \cdot m^2$$

$$EI_{yy} := E_{xx} \cdot I_{yy} = 8.196 \times 10^3 \frac{1}{m} \cdot kN \cdot m^2$$

FRP profile with coordinate system



Steel properties

Assume steel S355,

$f_y := 355MPa$

$E_{st} := 210GPa$

Loads

Selfweight

$g_{FRP} := 2 \cdot 103.69 \cdot g \frac{kg}{m^2} = 2.034 \cdot \frac{kN}{m^2}$ weight FRP (value for 2 decks on top of each other

$\rho_{steel} := 7800 \frac{kg}{m^3}$

$g_{steel} := \rho_{steel} g = 76.492 \cdot \frac{kN}{m^3}$ weight steel

$g_{asp} := 23 \frac{kN}{m^3}$ weight asphalt

$g_{as} := g_{asp} \cdot 0.1m = 2.3 \cdot \frac{kN}{m^2}$ assumed asphalt layer 1 dm

$g_{par} := 2 \frac{kN}{m}$ weight parapet

In ULS:

$g_{FRPuls} := g_{FRP} \cdot 1.35 = 2.745 \cdot \frac{kN}{m^2}$

$g_{aspuls} := g_{as} \cdot 1.35 = 3.105 \cdot \frac{kN}{m^2}$

$g_{paruls} := g_{par} \cdot 1.35 = 2.7 m \cdot \frac{kN}{m^2}$

Traffic load according to LM1 in Eurocode with additions from Trafikverket

$q_{l1} := 9 \frac{kN}{m}$ uniformly distributed loads for lane 1-4 and remaining areas

$q_{l2} := 2.5 \frac{kN}{m}$

$q_{l3} := q_{l2} = 2.5 \cdot \frac{kN}{m}$

$q_{l4} := q_{l2} = 2.5 \cdot \frac{kN}{m}$

$q_{re} := q_{l2} = 2.5 \cdot \frac{kN}{m}$

$Q_1 := 150kN$ point loads, half axle load (i.e. one wheel) for lane 1-3

$Q_2 := 100 \cdot kN$

$Q_3 := 50kN$

The traffic loads can be reduced according to Swedish National annex to Eurocode (from Trafikverket)

$\alpha_{q1} := 0.7$ reduction factors for uniformly distributed loads

$\alpha_{qi} := 1$

$\alpha_{qr} := 1$

$\alpha_{Q1} := 0.9$ reduction factors for point loads

$\alpha_{Q2} := \alpha_{Q1}$

$\alpha_{Q3} := 0$

$q_{l1red} := q_{l1} \cdot \alpha_{q1} = 6.3 \cdot \frac{kN}{m}$ reduced uniformly distributed loads

$q_{lired} := q_{l2} \cdot \alpha_{qi} = 2.5 \cdot \frac{kN}{m}$

$q_{rered} := q_{re} \cdot \alpha_{qr} = 2.5 \cdot \frac{kN}{m}$

$Q_{1red} := Q_1 \cdot \alpha_{Q1} = 1.35 \times 10^5 N$ reduced point loads

$Q_{2red} := Q_2 \cdot \alpha_{Q2} = 90 \cdot kN$

$Q_{3red} := Q_3 \cdot \alpha_{Q3} = 0 \cdot kN$ All loads above are applicable in SLS

Loads in ULS

$q_{l1ULS} := q_{l1red} \cdot 1.5 = 9.45 \cdot \frac{kN}{m}$

$q_{l2ULS} := q_{lired} \cdot 1.5 = 3.75 \cdot \frac{kN}{m}$

$q_{l3ULS} := q_{lired} \cdot 1.5 = 3.75 \cdot \frac{kN}{m}$

$q_{l4ULS} := q_{lired} \cdot 1.5 = 3.75 \cdot \frac{kN}{m}$

$q_{reULS} := q_{rered} \cdot 1.5 = 3.75 \cdot \frac{kN}{m}$

$Q_{1ULS} := Q_{1red} \cdot 1.5 = 202.5 \cdot kN$

$Q_{2ULS} := Q_{2red} \cdot 1.5 = 135 \cdot kN$

$Q_{3ULS} := Q_{3red} \cdot 1.5 = 0$

Application of loads

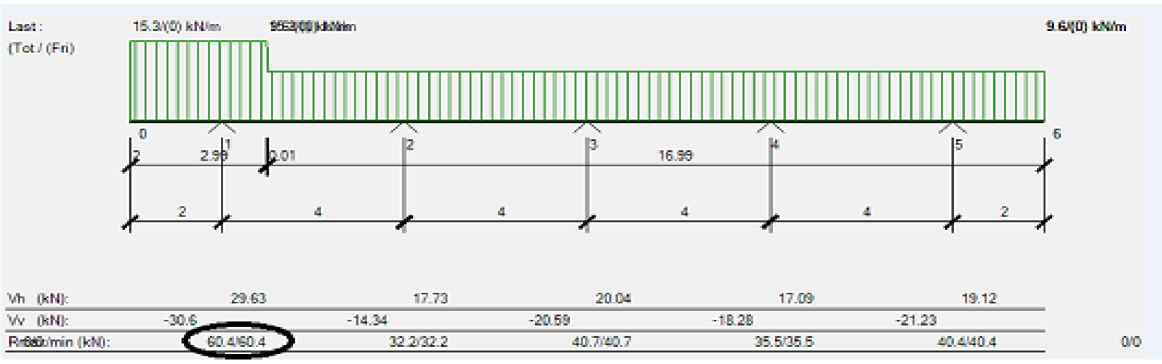
These loads were applied to a model made in the software SBBalk. A section of the bridge was modelled as a beam and then different scenarios were tried until the worst combination of lanes was found.
Uniformly distributed loads and point loads were modelled separately.
This resulted in support A getting the highest reaction force, the reaction force from the uniformly distributed loads was applied as a distributed load over the length of the beam and the reaction force from the point loads (the tandem system) was applied in two points (one for each axle).

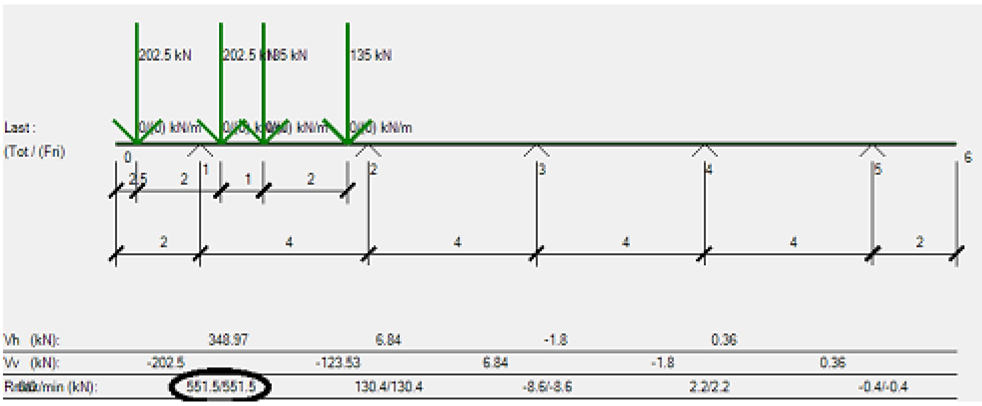
The point load was applied in the middle of the span when moment and deflection was evaluated and near the support when shear force was evaluated.

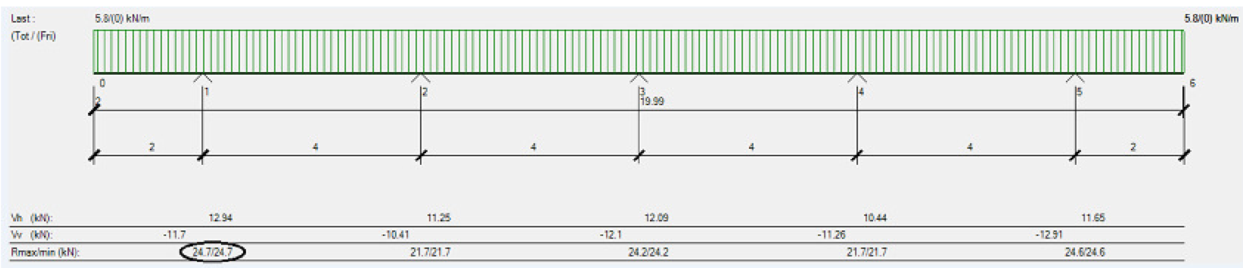
$q_A := 60.4 \frac{kN}{m}$ $\qquad g_A := 24.7 \frac{kN}{m}$

$P_A := 551.5kN$

Thus, we get a uniformly distributed load on R_A kN/m along the beam, named q_A and two point loads P_A from the axles. See files: sektion dist ULS.sbb, sektion point ULS.sbb, sektion self ULS.sbb.







Check moment, deflection and shear force

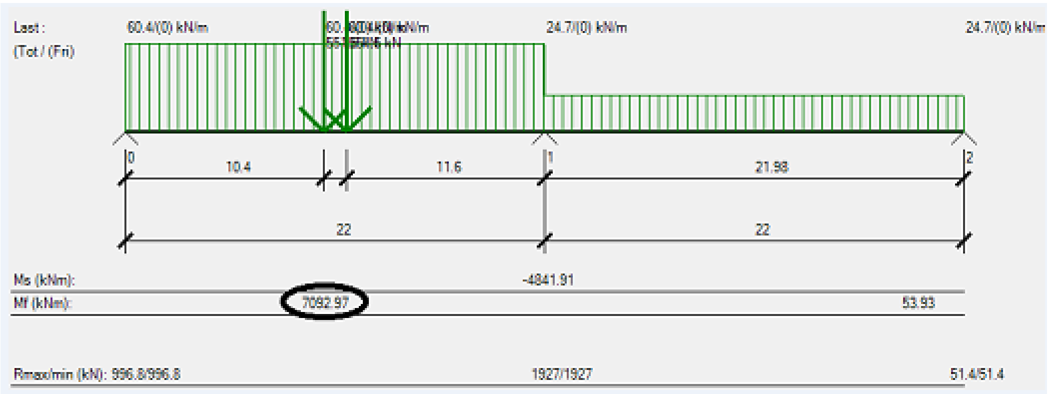
$$M_{Rd} := \frac{f_y \cdot I_{st}}{z} \qquad V_{Rd} := \frac{f_y \cdot I_{st} \cdot t_{web}}{S_{steel}}$$

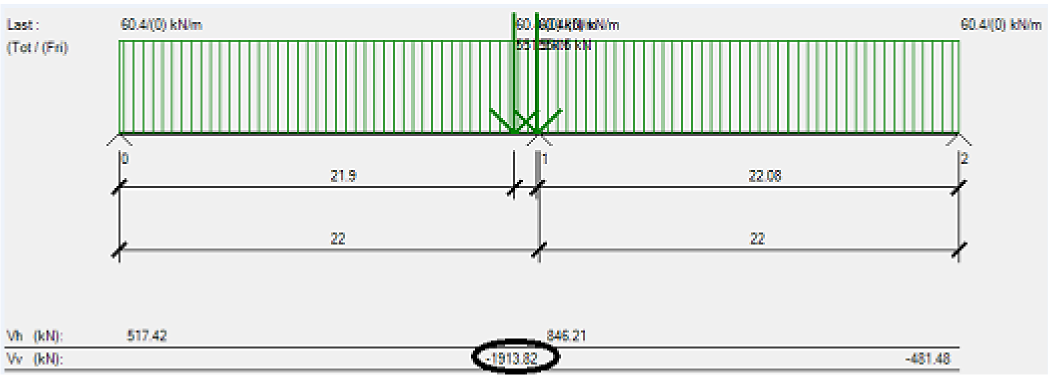
Design moment and shear force from SBBalk:
maximum moment is taken with two pointloads, c-c1.2 m centered around midspan. Maximum shear force is taken with the loads just next to the support. Values can be found in sbbalk-files: elevation ULS moment.sbb and elevation ULS shear.sbb.
The values are calculated using ULS values.

$$M_{sed} := -5921.83\mathrm{kN \cdot m}$$

$$M_{fed} := 7092.97\mathrm{kN \cdot m}$$

$$V_{sed} := 1913.82\mathrm{kN}$$





Create a I-section that fulfills the demands for moment and shear force:

$$h_{st} := 1000\mathrm{mm}$$

$$z := \frac{h_{st}}{2} = 0.5\,\mathrm{m}$$

$$t_{web} := 25\mathrm{mm}$$

$$t_{fl} := 39\mathrm{mm}$$

$$b_{fl} := 850\mathrm{mm}$$

$$h_{web} := h_{st} - 2 \cdot t_{fl} = 0.922\,\mathrm{m}$$

$$S_{steel} := t_{fl} \cdot b_{fl} \cdot \left(\frac{h_{st}}{2} - \frac{t_{fl}}{2} \right) + t_{web} \cdot \frac{\left(\frac{h_{st}}{2} - t_{fl} \right)^2}{2} = 0.019 \cdot m^3$$

$$I_{st} := \frac{h_{web}^{\ 3} \cdot t_{web}}{12} + 2 \cdot \frac{b_{fl} \cdot t_{fl}^{\ 3}}{12} + 2 b_{fl} \cdot t_{fl} \cdot \left(\frac{h_{st}}{2} - \frac{t_{fl}}{2} \right)^2 = 1.695 \times 10^{10} \cdot mm^4$$

$$M_{Rd} := \frac{f_y \cdot I_{st}}{z} = 1.203 \times 10^4 \cdot kN \cdot m \qquad V_{Rd} := \frac{f_y \cdot I_{st} \cdot t_{web}}{S_{steel}} = 8.094 \times 10^3 \cdot kN$$

$$M_{Rd} > \left| M_{fed} \right| = 1 \qquad V_{Rd} > V_{sed} = 1$$

Utilization rate for moment and shear force:

$$\frac{\left| M_{fed} \right|}{M_{Rd}} = 0.589 \qquad \frac{V_{sed}}{V_{Rd}} = 0.236$$

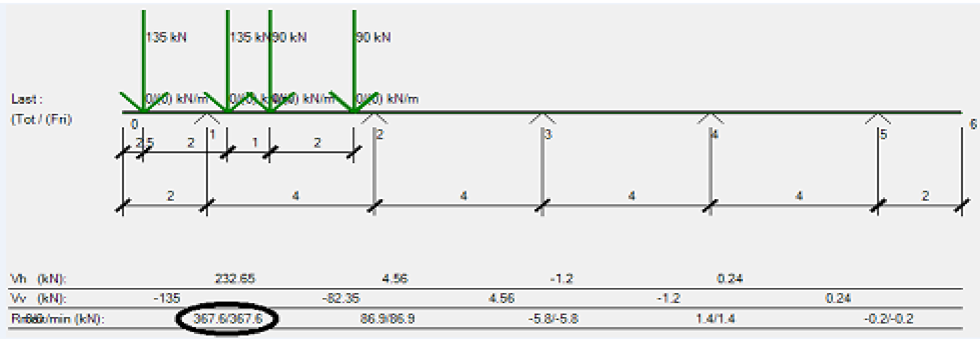
It works! :)

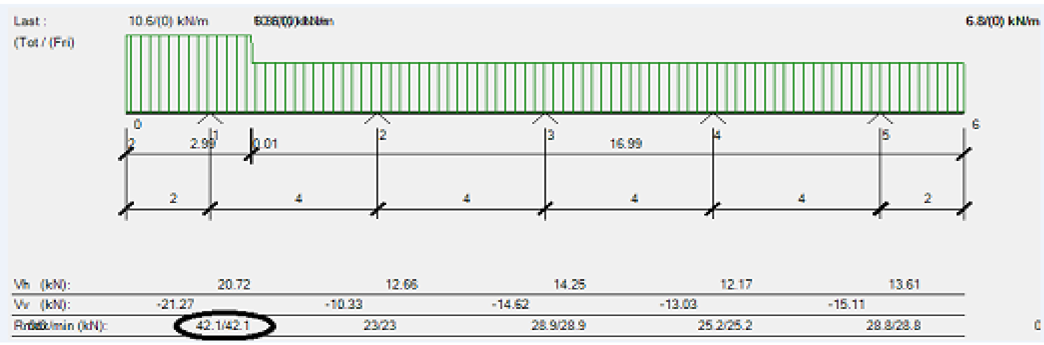
$$E_{st} \cdot I_{st} = 3.559 \times 10^9 \cdot N \cdot m^2$$

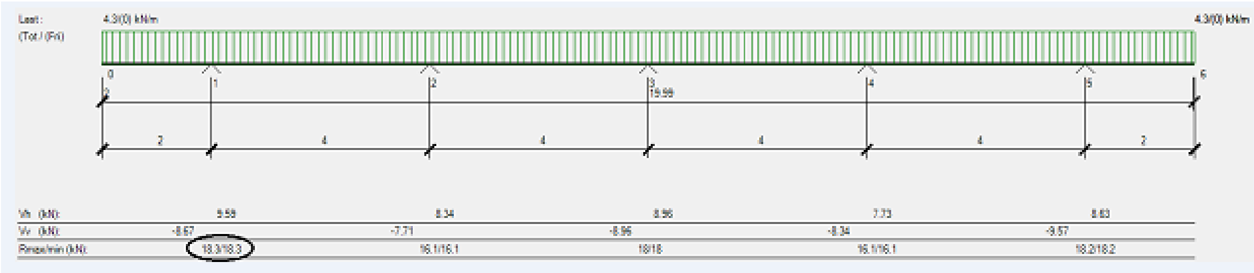
SBBalk shows which flexural rigidity (EI) is needed to get a deflection within limits. The I-section is adapted until it gets an EI large enough. This time, SLS values found in SBBalk are used (files: sektion dist SLS.sbb, sektion point SLS.sbb, sektion self SLS.sbb).

$$P_{ASLS} := 367.6 kN \qquad g_{ASLS} := 18.3 \frac{kN}{m}$$

$$q_{ASLS} := 42.1 \frac{kN}{m}$$







Limiting deflection is:

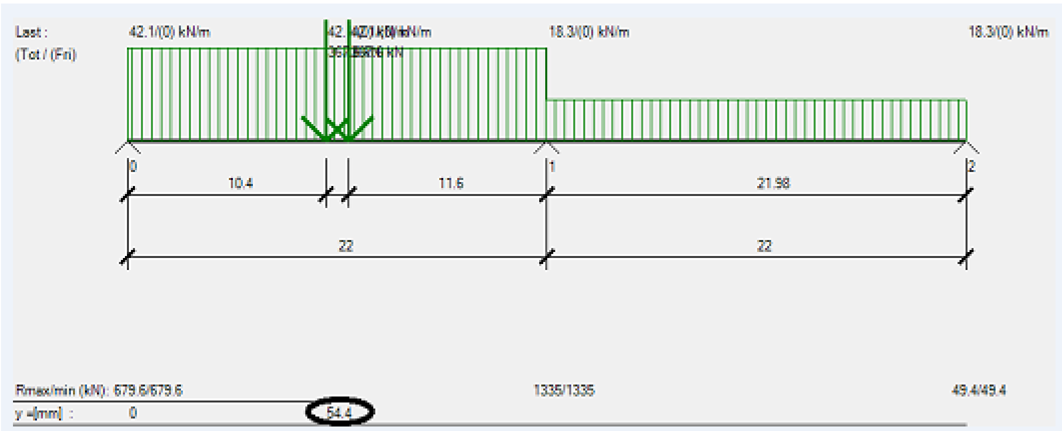
$$\delta_{max2} := \frac{L}{400} = 0.055\,\mathrm{m}$$

To achieve this we need an EI>3.5e9 Nm^2 according to SBBalk (see file elevation SLS.sbb).
Thus we need:

$$EI_b := 3.50 \cdot 10^9 N \cdot m^2$$

$$I_b := \frac{EI_b}{E_{st}} = 1.667 \times 10^{10} \cdot mm^4$$ needed value of I

$$I_{st} > I_b = 1 \qquad \frac{I_b}{I_{st}} = 0.983$$



Weight of beams:

$n_{beam} := 5$ number of longitudinal beams

$A_{st} := h_{web} \cdot t_{web} + 2t_{fl} \cdot b_{fl} = 8.935 \times 10^4 \cdot mm^2$ area of the cross-section of the I-beam

$V_{st} := A_{st} \cdot L \cdot n_{span} = 3.931 \cdot m^3$ volume of one beam

$A_{crfield} := 0.014m^2$ crossings are assumed to be the same as in the steel/concrete composite bridge

$A_{cross} := 0.018 \cdot m^2$

$l_{cross} := C_c = 4\,m$

$$P_{cross} := A_{crfield} \cdot \frac{l_{cross}}{2} \cdot \rho_{steel} \cdot g = 2.142 \cdot kN$$

the weight/force form the cross beams has not been included. As can be seen, the load from them is negligible

$$n_{cross} := 3 \cdot 7 = 21$$

number of cross beams

$$m_{st} := V_{st} \cdot \rho_{steel} \cdot n_{beam} + A_{cross} \cdot l_{cross} \cdot n_{cross} \cdot \rho_{steel} = 165.118 \cdot tonne$$

total weight of steel beams and crossings

$$m_2 := \frac{m_{st}}{L \cdot n_{span}} = 3.753 \cdot \frac{tonne}{m}$$

steel weight/meter

Exposed steel area that will need maintenance in the future:

$$A_{explong} := n_{beam} \cdot L \cdot n_{span} \cdot \left(2 \cdot h_{web} + 3 \cdot b_{fl}\right) = 966.68\, m^2$$

$$h_{webcross} := 778.6mm$$

$$b_{flcross} := 257mm$$

$$A_{expcross} := n_{cross} \cdot l_{cross} \cdot \left(2 \cdot h_{webcross} + 4 \cdot b_{flcross}\right) = 217.157\, m^2$$

$$A_{paint3} := A_{explong} + A_{expcross} = 1.184 \times 10^3\, m^2$$

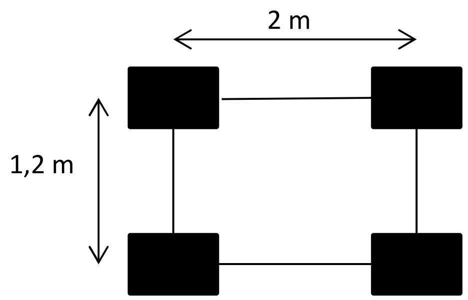
The analysis above is made considering the FRPdeck on rigid supports (the beams). In reality the "supports" will be flexible and a load distribution could be accounted for.

Appendix G

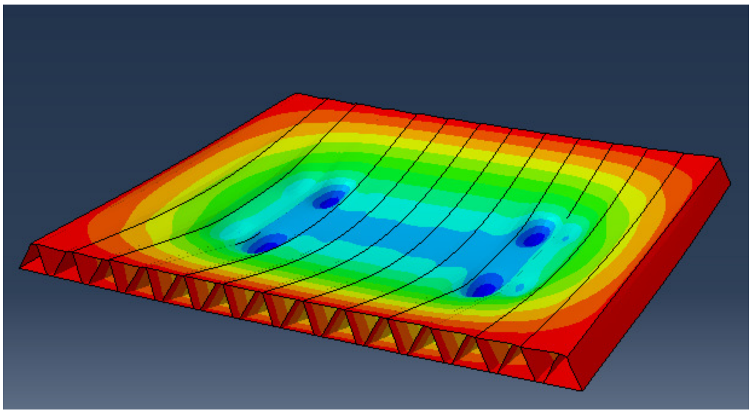
Deflection of FRP deck – FRP bridge alternative 2

A FEM-model of the FRP deck, simply supported on four edges, was created in Abaqus CAE 6.12-1 to check the deflection of the deck. The edges represents the steel girders and crossbeams and the size of the model is 4x4 meters.

The applied load was four wheel loads of 150 kN, representing double axles of a truck according to LM1 in Eurocode 1. The wheel load was reduced by a factor 0.9 and distributed on an area of 0.4x0.4 m, also according to Eurocode. The distance between the wheel loads are as below:



The limit for deflection of the deck is L/300, i.e. 4000mm/300=13.33mm. The FEM model showed that the maximum deflection on the bottom side of the deck was 12.6 mm, see the figure below. Hence, the deflection limit was fulfilled.



In the real situation, the deck will be continuous over the beams instead of simply supported as in this FEM model. This will result in lower deflections than those obtained above. Calculations were computed to find a relation between the deflection of a simply supported beam and a continuous beam. Based on this, an estimation was made that in the case of a continuous deck, the “plate” size could increase to 3.67m x 4.67m without compromising with the deflection limit (i.e. 3.67 m between cross beams and 4.67 m between main girders).

Appendix H

LCCA for the different alternatives

The alternatives are presented as follows:

- Steel/concrete alternative
- Steel/concrete alternative with deck change
- FRP alternative 1
- FRP alternative 1 with deck change
- FRP alternative 2
- FRP alternative 2 with deck change
- FRP alternative 3
- FRP alternative 3 with deck change
- Steel sandwich alternative

Life cycle cost analysis

General conditions

Name of project:	Ullevibron 14-1531-1	
Date:	2013-03-27	
Service life	years	100
Real discount rate	%	3,5%
ADT on bridge	veh/day	19 715
Percentage of trucks on bridge	%	5,1%
Passenger cars on bridge	veh/day	18 710
Trucks on bridge	veh/day	1 005
Allowed speed on bridge	km/h	50
Reduced speed on bridge	km/h	
ADT under bridge	veh/day	87 120
Percentage of trucks under bridge	%	8,2%
Passenger cars under bridge	veh/day	79 976
Trucks under bridge	veh/day	7 144
Allowed speed under bridge	km/h	70
Reduced speed under bridge	km/h	
Percentage night traffic, pc	%	11%
Percentage night traffic, trucks	%	16%
Hourly cost, car	SEK/h	167
Hourly cost, truck	SEK/h	347
Bridge length	m	44
Bridge width	m	20
Effective bridge width	m	18
Bridge area	m^2	880
Area of surfacing	m^2	792
Painted area	m^2	919
Length of edge beams	m	88

Investment costs

	Unit price	
Formwork	550	SEK/m^2
Piles	5 000	SEK/m
Concrete	1 800	SEK/m^3
Steel	24 500	SEK/ton
Reinforcement	13 200	SEK/ton
Insulation + surfacing	1 160	SEK/m^2

(Daniel T, approx, vet inte om det innehåller arbete mm, borde inkludera maskiner)

Material costs

	piles [m]	formwork [m2]	concrete [m3]	reinf [ton]	steel [ton]	insulation [m2]	cost [SEK]
Piles	168						840 000
Main beams					110		2 690 100
Cross beams					9		210 700
Bridge deck		880	220	33			1 315 600
Edge beam			22	3			83 160
Overlay						792	918 720
						Total cost:	**6 058 280**

User cost new bridge

	construction time [h]	affected pc	affected trucks	reduced speed [km/h]	length of detour [m]	normal way [m]	cost [SEK]
Steel structure northbound	10	4 359	580	50	534	445	1 674
Steel structure southbound	10	4 359	580	40	1 043	616	6 689
						Total cost:	**8 363**

	User cost replacement of bridge						
	construction time [h]	affected pc	affected trucks	reduced speed [km/h]	length of detour [m]	normal way [m]	cost [SEK]
Steel structure E6 northbound	10	4 359	580	50	534	445	1 674
Steel structure bridge nl 50%	10	1 020	82	40	1 609	1 695	1 052
Steel structure bridge nl 25% Fr	10	510	41	55	2 977	1 695	1 101
Steel structure bridge nl 25% MK	10	510	41	65	4 153	2 837	584
Steel structure southbound	10	4 359	580	40	1 043	616	6 689
Steel structure brdige sl	10	2 039	163	40	1 255	511	3 502
Reinforcment nl 50%	336	2 675	144	40	1 609	1 695	88 373
Reinforcment nl 25% Fr	336	1 338	72	55	2 977	1 695	92 523
Reinforcment nl 25% MK	336	1 338	72	65	4 153	2 837	49 095
Reinforcment sl	336	13 359	718	40	1 255	511	734 501
Concrete casting nl 50%	696	2 675	144	40	1 609	1 695	183 059
Concrete casting nl 25% Fr	696	1 338	72	55	2 977	1 695	191 655
Concrete casting nl 25% MK	696	1 338	72	65	4 153	2 837	101 696
Concrete casting sl	696	13 359	718	40	1 255	511	1 521 467
Insulation nl 50%	120	2 675	144	40	1 609	1 695	31 562
Insulation nl 25% Fr	120	1 338	72	55	2 977	1 695	33 044
Insulation nl 25% MK	120	1 338	72	65	4 153	2 837	17 534
Insulation sl	120	13 359	718	40	1 255	511	262 322
Surfacing nl 50%	8	2 675	144	40	1 609	1 695	1 736
Surfacing nl 25% Fr	8	1 338	72	55	2 977	1 695	2 203
Surfacing nl 25% MK	8	1 338	72	65	4 153	2 837	1 477
Surfacing sl	8	13 359	718	40	1 255	511	17 488
						Total cost:	**3 344 337**

Average speed normal way [km/h]:	62
Average speed normal way MK [km/h	57

North and south lane are separated due to the different amount of traffic. 28,6% in north lane, 71,4% in south lane.

Maintenance & repair

Activity cost

	Activity	Interval [yr]	Cost/unit [SEK/unit]	Quantity	Cost each time	Total present cost
Edge beams	Change [m]	45	10 800	88	950 400	245 092
Overlay	Change of insulation [m2]	40	1 500	792	1 188 000	375 842
	Change of surfacing [m2]	10	600	792	475 200	954 658
Steel	Repainting [m2]	30	1 700	919	1 562 300	825 577
Disposal	Edge beam	45	924	58	53 869	13 892
	Surfacing	10	40	224	8 960	20 835
					Total cost:	**2 435 896**

User cost

	Activity	Interval [yr]	disruption time [h]	Affected pc	Affected trucks	reduced speed [km/h]	length of detour [m	normal way [m]	cost [SEK]	Present cost
Edge beam	Change, nothern lane	45	1 092	5 351	288	30	200	200	142 734	36 809
	Change, southern lane	45	1 092	13 359	718	30	200	200	300 907	77 599
Overlay	Change of insulation, nl 50%	40	252	2 675	144	40	1 609	1 695	66 280	20 969
	Change of insulation, nl 25% Fr	40	252	1 338	72	55	2 977	1 695	69 392	21 953
	Change of insulation, nl 25% MK	40	252	1 338	72	65	4 153	2 837	36 821	11 649
	Change of insulation,sl	40	252	13 359	718	40	1 255	511	550 876	174 278
	Change of surfacing, nl 50%	10	10	292	23	40	1 609	1 695	301	700
	Change of surfacing, nl 25% Fr	10	10	146	12	55	2 977	1 695	315	732
	Change of surfacing, nl 25% MK	10	10	146	12	65	4 153	2 837	167	389
	Change of surfacing, sl	10	10	1 456	117	40	1 255	511	2 500	5 814
Steel	Repainting	30	20	4 359	580	50	200	200	885	468
									Total cost:	**351 358**

Average speed normal way nl [km/h]: 62
Average speed normal way nl MK [km/h]: 57

End-of-life

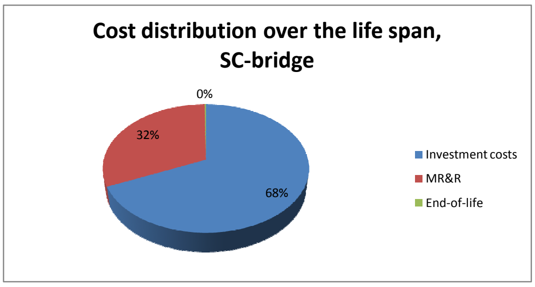
	Cost per tonne [SEK]	Quantity [tonne]	Cost [SEK]	Present value [SEK]
Concrete	1 100	641	705 430	22 616
Steel	-500	118	-59 200	-1 898
Asphalt	40	224	8 960	287
			Total cost	**21 005**

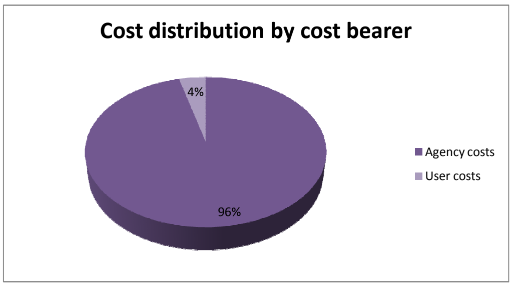
Only recycling/disposal costs for material is considered.
Costs for machines etc are not inclued.

Results

New bridge

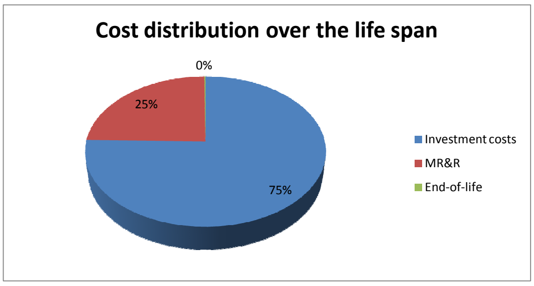
	Agency costs	User costs	Total present cost:
Investment costs	6 058 280	8 363	**6 066 643**
MR&R	2 435 896	351 358	**2 787 254**
End-of-life	21 005		**21 005**
Sum	**8 515 181**	**359 721**	**8 874 903**

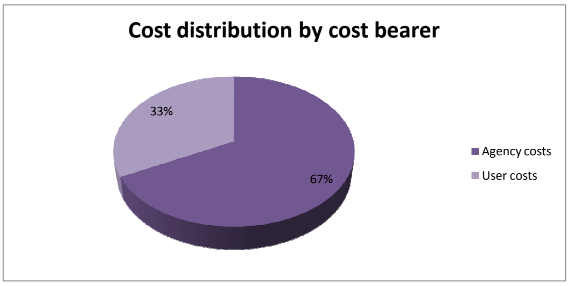




Replacement of bridge

	Agency costs	User costs	Total present cost:
Investment costs	5 218 280	3 344 337	**8 562 617**
MR&R	2 435 896	351 358	**2 787 254**
End-of-life	21 005		**21 005**
Sum	**7 675 181**	**3 695 695**	**11 370 877**





Appendix H - LCC Ullevi steel/concrete bridge with deck change

MR R

Same conditions as for the previous bridge but with a deck change after 50 years

Maintenance & repair

Activity cost

	Activity	Interval [yr]	Cost/unit [SEK/unit]	Quantity	Cost each time	Total present cost	
Deck	Change [m2]	50	5 700	880	5 016 000	898 132	aprice from BatMan "platta - betonrep. >110mm. Change of edge bemas excl
Overlay	Change of insulation [m2]	50	1 500	792	1 188 000	266 441	changed after 50 years, when the deck is changed, next change after 90 years
	Change of surfacing [m2]	10	600	792	475 200	954 658	
Steel	Repainting [m2]	30	1 700	919	1 562 300	825 577	
Disposal	Deck, concrete	50	1 100	605	665 500	140 496	
	Deck, reinforcement	50	-500	36	-18 150	-3 250	
	Surfacing	10	40	224	8 960	20 835	
					Total cost:	**3 102 889**	

User cost

	Activity	Interval [yr]	disruption time [h]	Affected pc	Affected trucks	reduced speed [km/h]	length of detour [m	normal way [m]	cost [SEK]	Present cost
Deck change	Formwork nl, 50%	50	168	2 675	144	40	1 609	1 695	44 187	7 912
	Formwork nl, 25% Fr	50	168	1 338	72	55	2 977	1 695	46 261	8 283
	Formwork nl, 25% MK	50	168	1 338	72	65	4 153	2 837	24 547	4 395
	Formwork sl	50	168	13 359	718	40	1 255	511	367 251	65 757
	Reinforcment nl 50%	50	336	2 675	144	40	1 609	1 695	88 373	15 824
	Reinforcment nl 25% Fr	50	336	1 338	72	55	2 977	1 695	92 523	16 567
	Reinforcment nl 25% MK	50	336	1 338	72	65	4 153	2 837	49 095	8 791
	Reinforcment sl	50	336	13 359	718	40	1 255	511	734 501	131 515
	Concrete casting nl 50%	50	696	2 675	144	40	1 609	1 695	183 059	32 777
	Concrete casting nl 25% Fr	50	696	1 338	72	55	2 977	1 695	191 655	34 316
	Concrete casting nl 25% MK	50	696	1 338	72	65	4 153	2 837	101 696	18 209
	Concrete casting sl	50	696	13 359	718	40	1 255	511	1 521 467	272 424
Overlay	Change of insulation, nl 50%	50	120	2 675	144	40	1 609	1 695	31 562	7 079
	Change of insulation, nl 25% Fr	50	120	1 338	72	55	2 977	1 695	33 044	7 411
	Change of insulation, nl 25% MK	50	120	1 338	72	65	4 153	2 837	17 534	3 932
	Change of insulation,sl	50	120	13 359	718	40	1 255	511	262 322	58 833
	Change of surfacing, nl 50%	10	10	292	23	40	1 609	1 695	301	700
	Change of surfacing, nl 25% Fr	10	10	146	12	55	2 977	1 695	315	732
	Change of surfacing, nl 25% MK	10	10	146	12	65	4 153	2 837	167	389
	Change of surfacing, sl	10	10	1 456	117	40	1 255	511	2 500	5 814
Steel	Repainting	30	20	4 359	580	50	200	200	885	468
									Total cost:	**702 127**

Average speed normal way nl [km/h]: 62

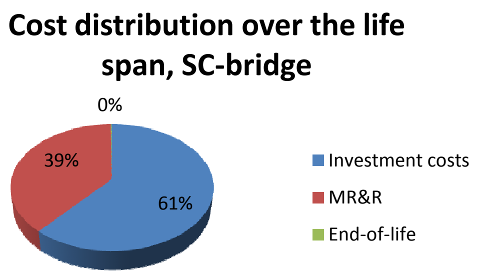
Average speed normal way nl MK [km/h]: 57

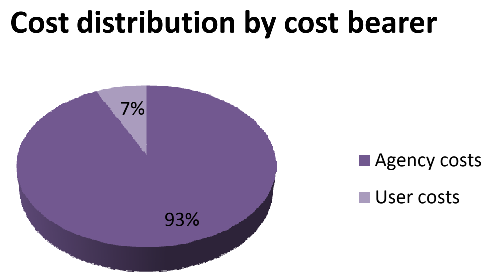
Same conditions as for the previous bridge but with a deck change after 50 years

Results

New bridge

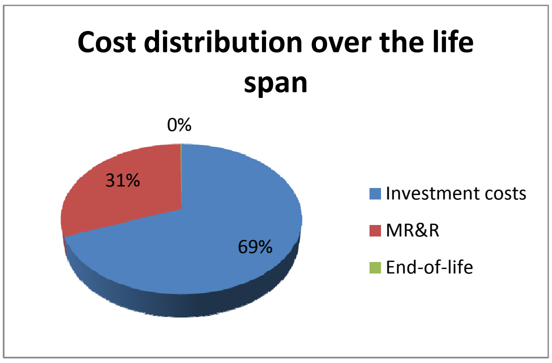
	Agency costs	User costs	Total present cost:
Investment costs	6 058 280	8 363	**6 066 643**
MR&R	3 102 889	702 127	**3 805 016**
End-of-life	19 143		**19 143**
Sum	**9 180 313**	**710 490**	**9 890 802**

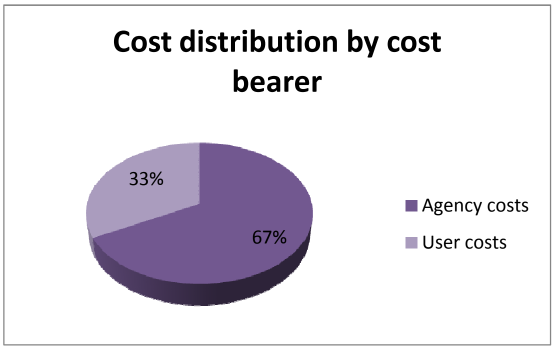




Replacement of bridge

	Agency costs	User costs	Total present cost:
Investment costs	5 218 280	3 344 337	**8 562 617**
MR&R	3 102 889	702 127	**3 805 016**
End-of-life	19 143		**19 143**
Sum	**8 340 313**	**4 046 464**	**12 386 776**





Life cycle cost analysis

General conditions

Name of project:	Alternative 1	
Date:	2013-04-02	
Service life	years	100
Real discount rate	%	3,5%
ADT on bridge	veh/day	19 715
Percentage of trucks on bridge	%	5,1%
Passenger cars on bridge	veh/day	18 710
Trucks on bridge	veh/day	1 005
Allowed speed on bridge	km/h	50
Reduced speed on bridge	km/h	
ADT under bridge	veh/day	87 120
Percentage of trucks under bridge	%	8,2%
Passenger cars under bridge	veh/day	79 976
Trucks under bridge	veh/day	7 144
Allowed speed under bridge	km/h	70
Reduced speed under bridge	km/h	
Percentage night traffic, pc	%	11%
Percentage night traffic, trucks	%	16%
Hourly cost, car	SEK/h	167
Hourly cost, truck	SEK/h	347
Bridge length	m	44
Bridge width	m	20
Effective bridge width	m	18
Bridge area	m^2	880
Area of surfacing	m^2	792
Painted area	m^2	1 222
Length of edge beams	m	88

Investment costs

	Unit price	
Piles	5 000	SEK/m
FRP Asset deck	6 338	SEK/m^2
Steel	24 500	SEK/ton
Surfacing	1 008	SEK/m^2

	Material costs				
	piles [m]	FRP [m2]	steel [ton]	surfacing [m2]	cost [SEK]
Piles	140				700 000
Main beams			110		2 704 800
Cross beams			11		269 500
Bridge deck		880			5 577 440
Edge beam		44			278 872
Overlay				792	798 336
				Total cost:	**10 328 948**

	User cost new bridge						
	construction time [h]	affected pc	affected trucks	reduced speed [km/h]	length of detour [m]	normal way [m]	cost [SEK]
Steel structure northbound	10	4 359	580	50	534	445	1 674
Steel structure southbound	10	4 359	580	40	1 043	616	6 689
						Total cost:	**8 363**

Assuming that lifting the steel structure can be done in 1 night/span

Appendix H - LCC FRP alternative 1

Investment costs

	User cost replacement of bridge						
	construction time [h]	affected pc	affected trucks	reduced speed [km/h]	length of detour [m]	normal way [m]	cost [SEK]
Steel structure E6 north	10	4 359	580	50	534	445	1 674
Steel structure bridge nl50%	10	1 020	82	40	1 609	1 695	1 052
Steel structure bridge nl 25% Fr	10	510	41	55	2 977	1 695	1 101
Steel structure bridge nl25% MK	10	510	41	65	4 153	2 837	584
Curing, nl 50%	4	2 384	120	40	1 609	1 695	932
Curing, nl 25% Fr	4	1 192	60	55	2 977	1 695	975
Curing, nl 25% MK	4	1 192	60	65	4 153	2 837	518
Curing, sl	4	11 903	601	40	1 255	511	7 744
Steel structure E6 south	10	4 359	580	40	1 043	616	6 689
Steel structure bridge sl	10	2 039	163	40	1 255	511	3 502
						Total cost:	**24 771**

Average speed normal way nl [km/h]:	62
Average speed normal way nl MK [km/h]	57

Maintenance & repair

Activity cost

	Activity	Interval [yr]	Cost/unit [SEK/unit]	Quantity	Cost each time	Total present cost
FRP	Reparation of eb/deck	20	78	10	781	739
Overlay	Change of surfacing [m2]	20	630	792	498 960	471 949
Steel	Repainting [m2]	30	1 700	1 222	2 077 400	1 097 774
Disposal	Surfacing	20	1 100	54	59 067	55 870
					Total cost:	**1 626 332**

User cost

	Activity	Interval [yr]	disruption time [h]	Affected pc	Affected trucks	reduced speed [km/h]	length of detour [m]	normal way [m]	cost [SEK]	Present cost
FRP	Reparations of FRP eb, nl	20	10	583	47	30	200	200	126	119
	Reparations of FRP eb, sl	20	10	1 456	117	30	200	200	315	298
Overlay	Change of surfacing, nl 50%	20	10	292	23	40	1 609	1 695	301	285
	Change of surfacing, nl 25% Fr	20	10	146	12	55	2 977	1 695	315	298
	Change of surfacing, nl 25% MK	20	10	146	12	65	4 153	2 837	167	158
	Change of surfacing, sl	20	10	1 456	117	40	1 255	511	2 500	2 365
Steel	Repainting	30	20	4 359	580	50	200	200	885	468
									Total cost:	**3 991**

Average speed normal way nl [km/h]	62
Average speed normal way nl MK [kı	57

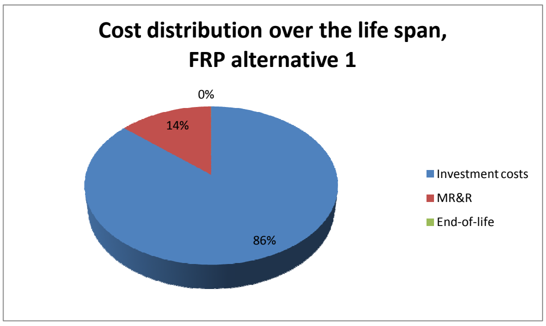
End-of-life

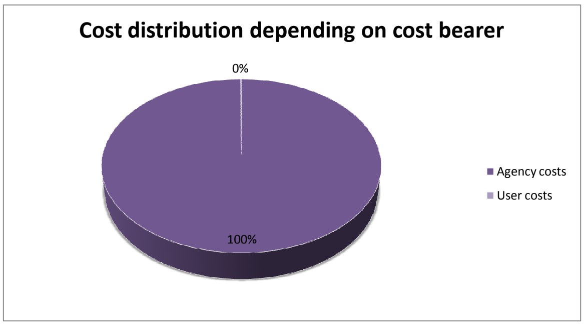
	Cost per unit [SEK]	Quantity [tonne]	Cost [SEK]	Present value [SEK]
FRP	1 100	96	105 391	3 379
Steel	-500	121	-60 700	-1 946
Polymer concrete	1 100	54	59 067	1 894
			Total cost	**3 326**

Results

New bridge

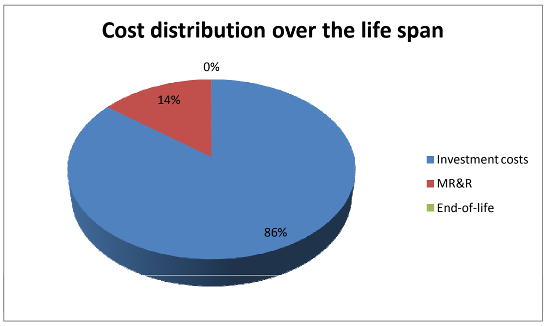
	Agency costs	User costs	Total present cost
Investment costs	10 328 948	8 363	**10 337 311**
MR&R	1 626 332	3 991	**1 630 323**
End-of-life	3 326		**3 326**
Sum	**11 958 607**	**12 354**	**11 970 960**

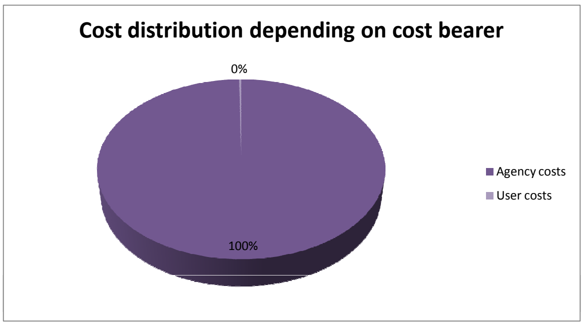




Replacement of bridge

	Agency costs	User costs	Total present cost
Investment costs	9 628 948	24 771	**9 653 719**
MR&R	1 626 332	3 991	**1 630 323**
End-of-life	3 326		**3 326**
Sum	**11 258 607**	**28 761**	**11 287 368**





Appendix H - LCC FRP alternative 1

MR R

Same conditions as for the previous bridge but with a deck change after 50 years

Maintenance & repair

Activity cost

	Activity	Interval [yr]	Cost/unit [SEK/unit]	Quantity	Cost each time	Total present cost
Overlay	Change of surfacing [m2]	20	630	792	498 960	471 949
Steel	Repainting [m2]	30	1 700	1 222	2 077 400	1 097 774
Deck	frp repl	80	6 338	880	5 577 440	355 801
Disposal	FRP	80	1 100	96	105 391	6 723
	Surfacing	20	1 100	54	59 067	55 870
					Total cost:	**1 988 117**

User cost

	Activity	Interval [yr]	disruption time [h]	Affected pc	Affected trucks	reduced speed [km/h]	length of detour [m]	normal way [m]	cost [SEK]	Present cost
Overlay	Change of surfacing, nl 50% [m2]	20	10	292	23	40	1 609	1 695	301	285
	Change of surfacing, nl 25% Fr [m2]	20	10	146	12	55	2 977	1 695	315	298
	Change of surfacing, nl 25% MK [m2]	20	10	146	12	65	4 153	2 837	167	158
	Change of surfacing, sl [m2]	20	10	1 456	117	40	1 255	511	2 500	2 365
Deck	FRP replacement, sl	80	10	1 456	117	40	1 255	511	2 500	170
	FRP replacement, nl 50%	80	10	292	23	40	1 609	1 695	301	19
	FRP replacement, nl 25% Fr	80	10	146	12	55	2 977	1 695	315	20
	FRP replacement, nl 25% MK	80	10	146	12	65	4 153	2 837	167	11
	FRP replacement E6 north	80	2	4 359	580	50	534	445	335	21
	FRP replacement E6 south	80	2	4 359	580	40	1 043	616	1 338	85
Steel	Repainting [m2]	30	20	4 359	580	50	200	200	885	468
									Total cost:	**3 899**

Average speed normal way nl [km/h]: 62

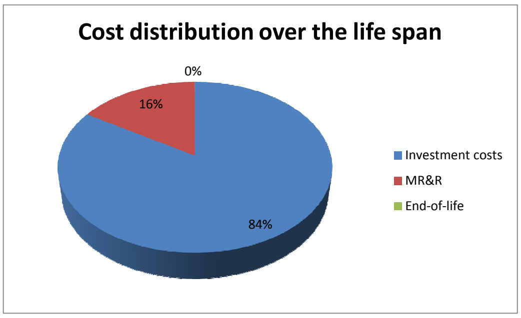
Average speed normal way nl MK [km/h]: 57

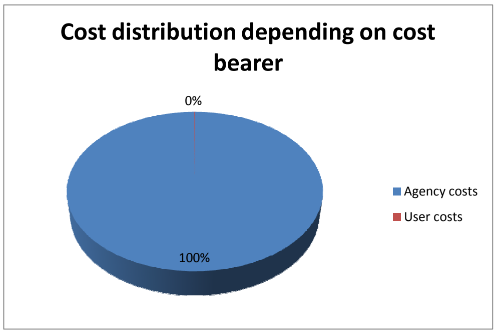
Same conditions as for the previous bridge but with a deck change after 50 years

Results

New bridge

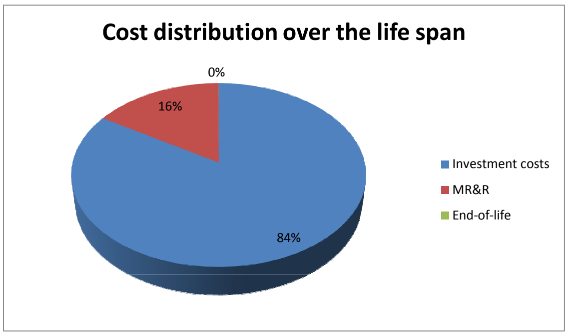
	Agency costs	User costs	Total present cost
Investment costs	10 328 948	8 363	**10 337 311**
MR&R	1 988 117	3 899	**1 992 017**
End-of-life	3 326		**3 326**
Sum	**12 320 392**	**12 262**	**12 332 654**

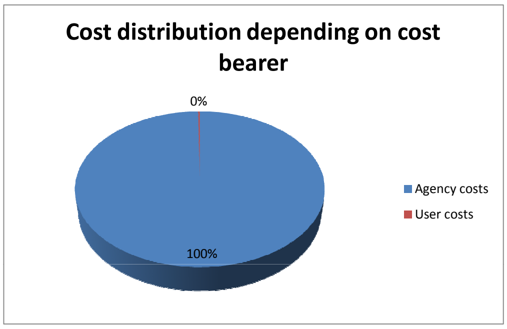




Replacement of bridge

	Agency costs	User costs	Total present cost
Investment costs	10 328 948	24 771	**10 353 719**
MR&R	1 988 117	3 899	**1 992 017**
End-of-life	3 326		**3 326**
Sum	**12 320 392**	**28 670**	**11 649 062**





Life cycle cost analysis

General conditions

Name of project:	Alternative 2	
Date:	2013-04-03	
Service life	years	100
Real discount rate	%	3,5%
ADT on bridge	veh/day	19 715
Percentage of trucks on bridge	%	5,1%
Passenger cars on bridge	veh/day	18 710
Trucks on bridge	veh/day	1 005
Allowed speed on bridge	km/h	50
Reduced speed on bridge	km/h	
ADT under bridge	veh/day	87 120
Percentage of trucks under bridge	%	8,2%
Passenger cars under bridge	veh/day	79 976
Trucks under bridge	veh/day	7 144
Allowed speed under bridge	km/h	70
Reduced speed under bridge	km/h	
Percentage during night, pc	%	11%
Percentage during night, trucks	%	16%
Hourly cost, car	SEK/h	167
Hourly cost, truck	SEK/h	347
Bridge length	m	44
Bridge width	m	20
Effective bridge width	m	18
Bridge area	m^2	880
Area of surfacing	m^2	792
Painted area	m^2	1 221
Length of edge beams	m	88

Investment costs

	Unit price	
Piles	5 000	SEK/m
FRP Asset deck	6 338	SEK/m^2
Steel	24 500	SEK/ton
IPE600	13 700	SEK/ton
Surfacing	1 008	SEK/m^2

Material costs

	piles [m]	FRP [m2]	steel [ton]	surfacing [m2]	cost [SEK]
Piles	140				700 000
Main beams			101,4		2 484 300
Cross beams			24,4		334 499
Bridge deck		880			5 577 440
Edge beam		44			278 872
Overlay				792	798 336
				Total cost:	**10 173 447**

User cost new bridge

	construction time [h]	affected pc	affected trucks	reduced speed [km/h]	length of detour [m]	normal way [m]	cost [SEK]
Steel structure northbound	10	4 359	580	50	534	445	1 674
Steel structure southbound	10	4 359	580	40	1 043	616	6 689
						Total cost:	**8 363**

Appendix H - LCC FRP alternative 2

Investment costs

	User cost replacement of bridge						
	construction time [h]	affected pc	affected trucks	reduced speed [km/h]	length of detour [m]	normal way [m]	cost [SEK]
Steel structure E6 northbound	10	4 359	580	50	534	445	1 674
Steel structure bridge nl50%	10	1 020	82	40	1 609	1 695	1 052
Steel structure bridge nl 25% Fr	10	510	41	55	2 977	1 695	1 101
Steel structure bridge nl25% MK	10	510	41	65	4 153	2 837	584
Curing, nl 50%	4	2 384	120	40	1 609	1 695	932
Curing, nl 25% Fr	4	1 192	60	55	2 977	1 695	975
Curing, nl 25% MK	4	1 192	60	65	4 153	2 837	518
Curing pc, sl	4	11 903	601	40	1 255	511	7 744
Steel strucutre E6 southbound	10	4 359	580	40	1 043	616	6 689
Steel structure bridge sl	10	2 039	163	40	1 255	511	3 502
						Total cost:	**24 771**

Average speed normal way [km/h]: 62

Average speed normal way MK [km/h] 57

Maintenance & repair

Activity cost

	Activity	Interval [yr]	Cost/unit [SEK/unit]	Quantity	Cost each time	Total present cost
FRP	Reparation of eb/deck	20	78	10	781	739
Overlay	Change of surfacing [m2]	20	630	792	498 960	471 949
Steel	Repainting [m2]	30	1 700	1 221	2 075 700	1 096 876
Disposal	Surfacing	20	1 100	54	59 067	55 870
					Total cost:	**1 625 434**

User cost

	Activity	Interval [yr]	disruption time [h]	Affected pc	Affected trucks	reduced speed [km/h]	length of detour [m]	normal way [m]	cost [SEK]	Present cost
FRP	Reparations of FRP eb, nl	20	10	583	47	30	200	200	126	119
	Reparations of FRP eb, sl	20	10	1 456	117	30	200	200	315	298
Overlay	Change of surfacing, nl 50%[m2]	20	10	292	23	40	1 609	1 695	301	285
	Change of surfacing, nl 50%[m2]	20	10	146	12	55	2 977	1 695	315	298
	Change of surfacing, nl 25% MK[m2]	20	10	146	12	65	4 153	2 837	167	158
	Change of surfacing, sl [m2]	20	10	1 456	117	40	1 255	511	2 500	2 365
Steel	Repainting [m2]	30	20	4 359	580	50	200	200	885	468
									Total cost:	**3 991**

Average speed normal way nl [km/h]: 62
Average speed normal way nl MK [km/h]: 57

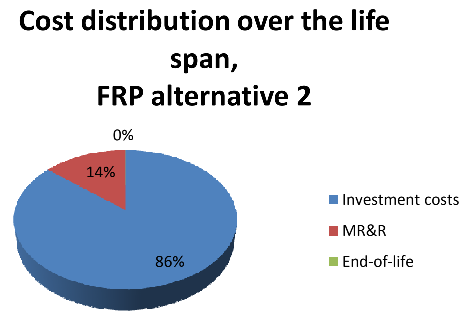
End-of-life

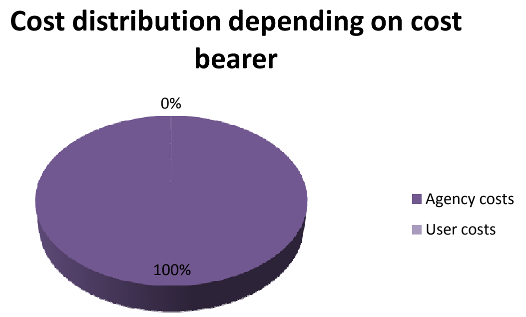
	Cost per unit [SEK]	Quantity [tonne]	Cost [SEK]	Present value [SEK]
FRP	1 100	96	105 391	3 379
Steel	-500	126	-62 908	-2 017
Polymer concrete	1 100	54	59 067	1 894
			Total cost	**3 256**

Results

New bridge

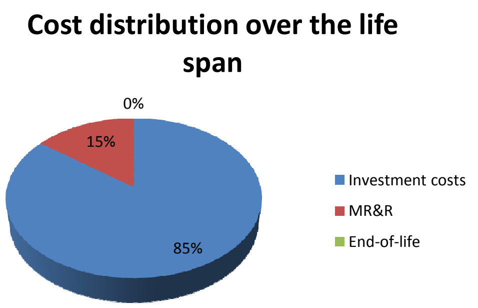
	Agency costs	User costs	Total present cost
Investment costs	10 173 447	8 363	**10 181 810**
MR&R	1 625 434	3 991	**1 629 424**
End-of-life	3 256		**3 256**
Sum	**11 802 137**	**12 354**	**11 814 490**

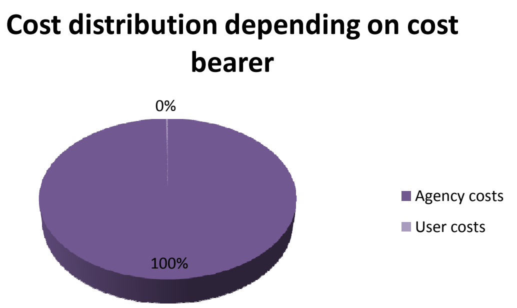




Replacement of bridge

	Agency costs	User costs	Total present cost
Investment costs	9 473 447	24 771	**9 498 218**
MR&R	1 625 434	3 991	**1 629 424**
End-of-life	3 256		**3 256**
Sum	**11 102 137**	**28 761**	**11 130 898**





Appendix H - LCC FRP alternative 2

MR R

Same conditions as for the previous bridge but with a deck change

Maintenance & repair

Activity cost

	Activity	Interval [yr]	Cost/unit [SEK/unit]	Quantity	Cost each time	Total present cost
Overlay	Change of surfacing [m2]	20	630	792	498 960	471 949
Steel	Repainting [m2]	30	1 700	1 221	2 075 700	1 096 876
Deck	frp repl	80	6 338	880	5 577 264	355 790
Disposal	FRP	80	1 100	96	105 391	6 723
	Surfacing	20	1 100	54	59 067	55 870
					Total cost:	**1 987 208**

User cost

	Activity	Interval [yr]	disruption time [h]	Affected pc	Affected trucks	reduced speed [km/h]	length of detour [m]	normal way [m]	cost [SEK]	Present cost
Overlay	Change of surfacing, nl 50%	20	10	292	23	40	1 609	1 695	301	285
	Change of surfacing, nl 25 Fr%	20	10	146	12	55	2 977	1 695	315	298
	Change of surfacing, nl 25% MK	20	10	146	12	65	4 153	2 837	167	158
	Change of surfacing, sl	20	10	1 456	117	40	1 255	511	2 500	2 365
Deck	FRP replacement, sl	80	10	1 456	117	40	1 255	511	2 500	159
	FRP replacement, nl 50%	80	10	292	23	40	1 609	1 695	301	19
	FRP replacement, nl 25% Fr	80	10	146	12	55	2 977	1 695	315	20
	FRP replacement, nl 25% MK	80	10	146	12	65	4 153	2 837	167	11
	FRP replacement E6 north	80	2	4 359	580	50	534	445	335	21
	FRP replacement E6 south	80	2	4 359	580	40	1 043	616	1 338	85
Steel	Repainting [m2]	30	20	4 359	580	50	200	200	885	468
									Total cost:	**3 889**

Average speed normal way nl [km/h]: 62

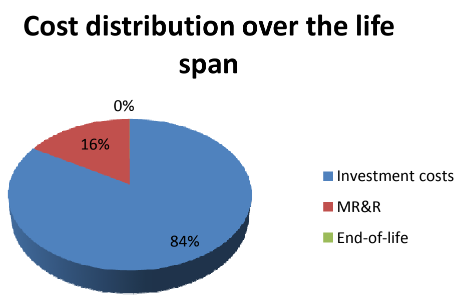
Average speed normal way nl MK [km/h]: 57

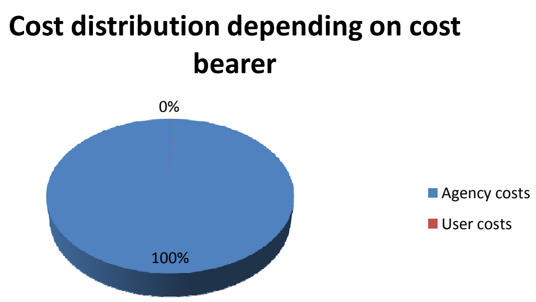
Same conditions as for the previous bridge but with a deck change

Results

New bridge

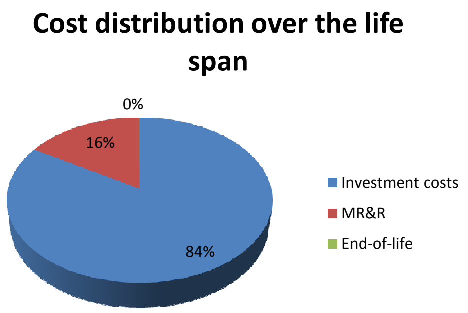
	Agency costs	User costs	Total present cost
Investment costs	10 173 262	8 363	**10 181 625**
MR&R	1 987 208	3 889	**1 991 097**
End-of-life	3 256		**3 256**
Sum	**12 163 726**	**12 252**	**12 175 978**

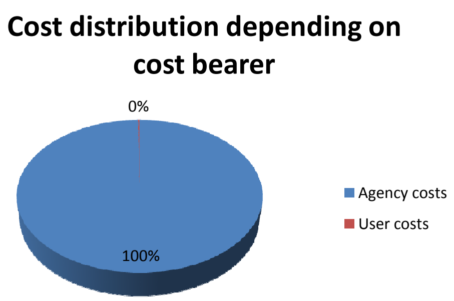




Replacement of bridge

	Agency costs	User costs	Total present cost
Investment costs	10 173 262	24 771	**10 198 033**
MR&R	1 987 208	3 889	**1 991 097**
End-of-life	3 256		**3 256**
Sum	**12 163 726**	**28 660**	**11 492 386**





Life cycle cost analysis

General conditions

Name of project:	Alternative 3	
Date:	2013-04-03	
Service life	years	100
Real discount rate	%	3,5%
ADT on bridge	veh/day	19 715
Percentage of trucks on bridge	%	5,1%
Passenger cars on bridge	veh/day	18 710
Trucks on bridge	veh/day	1 005
Allowed speed on bridge	km/h	50
Reduced speed on bridge	km/h	
ADT under bridge	veh/day	87 120
Percentage of trucks under bridge	%	8,2%
Passenger cars under bridge	veh/day	79 976
Trucks under bridge	veh/day	7 144
Allowed speed under bridge	km/h	70
Reduced speed under bridge	km/h	
Percentage night traffic, pc	%	10,9%
Percentage night traffic. trucks	%	16,3%
Hourly cost, car	SEK/h	167
Hourly cost, truck	SEK/h	347
Bridge length	m	44
Bridge width	m	20
Effective bridge width	m	18
Bridge area	m^2	880
Area of surfacing	m^2	792
Painted area	m^2	1 184
Length of edge beams	m	88

Investment costs

	Unit price	
Piles	5 000	SEK/m
FRP Asset deck	6 338	SEK/m^2
Steel	24 500	SEK/ton
Surfacing	1 008	SEK/m^2

	Material costs				
	piles [m]	FRP [m2]	steel [ton]	surfacing [m2]	cost [SEK]
Piles	168				840 000
Main beams			153,3		3 755 850
Cross beams			11,8		289 100
Bridge deck		1 760			11 154 880
Edge beam		44			278 872
Overlay				792	798 336
				Total cost:	**17 117 038**

	User cost						
	construction time [h]	affected pc	affected trucks	reduced speed [km/h]	length of detour [m]	normal way [m]	cost [SEK]
Steel structure northbound	10	4 359	580	50	534	445	1 674
Steel structure southbound	10	4 359	580	40	1 043	616	6 689
						Total cost:	**8 363**

Assuming that lifting the steel structure takes the same time as alt. 1 and 2

	User cost replacement of bridge						
	construction time [h]	affected pc	affected trucks	reduced speed [km/h]	length of detour [m]	normal way [m]	cost [SEK]
Steel structure E6 north	10	4 359	580	50	534	445	1 674
Steel structure bridge nl50%	10	1 020	82	40	1 609	1 695	1 052
Steel structure bridge nl 25% Fr	10	510	41	55	2 977	1 695	1 101
Steel structure bridge nl25% MK	10	510	41	65	4 153	2 837	584
Curing, nl 50%	4	2 384	120	40	1 609	1 695	932
Curing, nl 25% Fr	4	1 192	60	55	2 977	1 695	975
Curing, nl 25% MK	4	1 192	60	65	4 153	2 837	518
Curing, sl	4	11 903	601	40	1 255	511	7 744
Steel structure E6 south	10	4 359	580	40	1 043	616	6 689
Steel structure bridge south	10	2 039	163	40	1 255	511	3 502
						Total cost:	**24 771**

Average speed normal way [km/h]:	62
Average speed normal way MK [km/h	57

Maintenance & repair

Activity cost

	Activity	Interval [yr]	Cost/unit [SEK/unit]	Quantity	Cost each time	Total present cost
FRP	Reparation of eb/deck	20	78	10	781	739
Overlay	Change of surfacing [m2]	20	630	792	498 960	471 949
Steel	Repainting [m2]	30	1 700	1 184	2 012 800	1 063 637
Disposal	Surfacing	20	1 100	54	59 067	55 870
					Total cost:	**1 592 195**

User cost

	Activity	Interval [yr]	disruption time [h]	Affected pc	Affected trucks	reduced speed [km/h]	length of detour [m]	normal way [m]	cost [SEK]	Present cost
FRP	Reparations of FRP eb, nl	20	10	583	47	30	200	200	126	119
	Reparations of FRP eb, sl	20	10	1 456	117	30	200	200	315	298
Overlay	Change of surfacing, nl 50%	20	10	292	23	40	1 609	1 695	301	285
	Change of surfacing, nl 25% Fr	20	10	146	12	55	2 977	1 695	315	298
	Change of surfacing, nl 50% MK	20	10	146	12	65	4 153	2 837	167	158
	Change of surfacing, sl	20	10	1 456	117	40	1 255	511	2 500	2 365
Steel	Repainting	30	20	4 359	580	50	200	200	885	468
									Total cost:	**3 991**

Average speed normal way nl [km/h]: 62
Average speed normal way nl MK [km/h]: 57

End-of-life

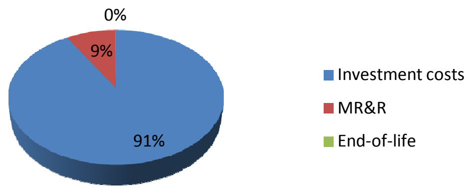
	Cost per unit [SEK]	Quantity [tonne]	Cost [SEK]	Present value [SEK]
FRP	1 100	187	205 762	6 597
Steel	500	165	82 550	2 647
Polymer concrete	1 100	54	59 067	1 894
			Total cost	**11 137**

Results

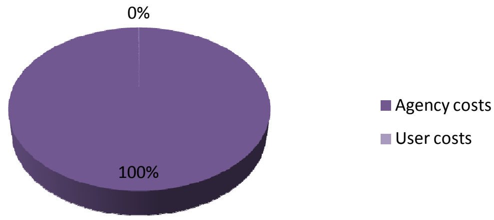
New bridge

	Agency costs	User costs	Total present cost
Investment costs	17 117 038	8 363	**17 125 401**
MR&R	1 592 195	3 991	**1 596 186**
End-of-life	11 137		**11 137**
Sum	**18 720 370**	**12 354**	**18 732 724**

Cost distribution over the life span, FRP alternative 3



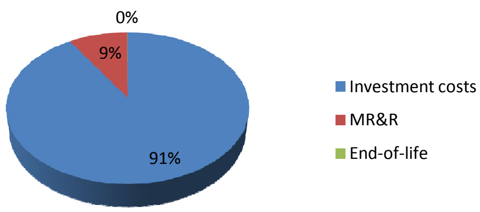
Cost distribution depending on cost bearer



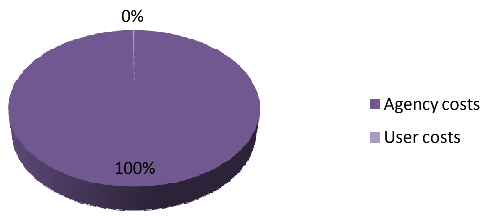
Replacement of bridge

	Agency costs	User costs	Total present cost
Investment costs	16 277 038	24 771	**16 301 809**
MR&R	1 592 195	3 991	**1 596 186**
End-of-life	11 137	0	**11 137**
Sum	**17 880 370**	**28 761**	**17 909 132**

Cost distribution over the life span



Cost distribution depending on cost bearer



Appendix H - LCC FRP alternative 3

MR R

Same conditions as for the previous bridge but with a deck change

Maintenance & repair

Activity cost

	Activity	Interval [yr]	Cost/unit [SEK/unit]	Quantity	Cost each time	Total present cost
Overlay	Change of surfacing [m2]	20	630	792	498 960	471 949
Steel	Repainting [m2]	30	1 700	1 184	2 012 800	1 063 637
Deck	frp repl	35	6 338	1 760	11 154 880	4 349 990
Disposal	FRP	35	1 100	187	205 762	80 240
	Surfacing	20	1 100	54	59 067	55 870
					Total cost:	**6 021 686**

User cost

	Activity	Interval [yr]	disruption time [h]	Affected pc	Affected trucks	reduced speed [km/h]	length of detour [m]	normal way [m]	cost [SEK]	Present cost
Overlay	Change of surfacing, nl 50%	20	10	292	23	40	1 609	1 695	301	285
	Change of surfacing, nl 25% Fr	20	10	146	12	55	2 977	1 695	315	298
	Change of surfacing, nl 50% MK	20	10	146	12	65	4 153	2 837	167	158
	Change of surfacing, sl	20	10	1 456	117	40	1 255	511	2 500	2 365
Deck	FRP replacement, sl	35	10	1 456	117	40	1 255	511	2 500	975
	FRP replacement, nl 50%	35	10	292	23	40	1 609	1 695	301	117
	FRP replacement, nl 25% Fr	35	10	146	12	55	2 977	1 695	315	123
	FRP replacement, nl 25% MK	35	10	146	12	65	4 153	2 837	167	65
	FRP replacement E6 north	35	2	4 359	580	50	534	445	335	131
	FRP replacement E6 south	35	2	4 359	580	40	1 043	616	1 338	522
Steel	Repainting [m2]	30	20	4 359	580	50	200	200	885	468
									Total cost:	**5 506**

Average speed normal way nl [km/h]:	62
Average speed normal way nl MK [km/h]:	57

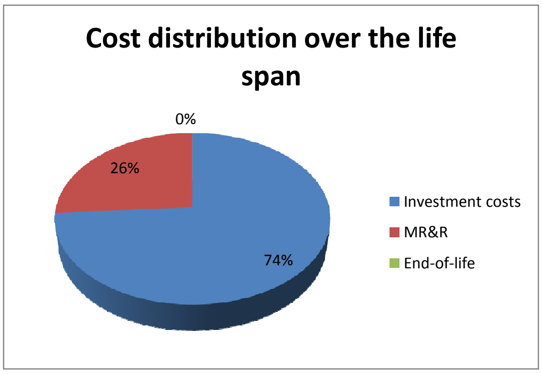
Results

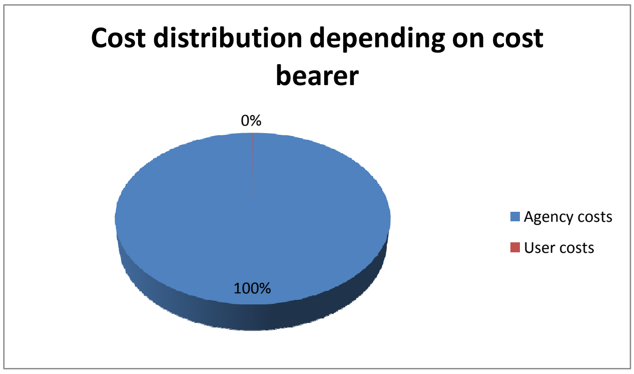
Same conditions as for the previous bridge but with a deck change

Results

New bridge

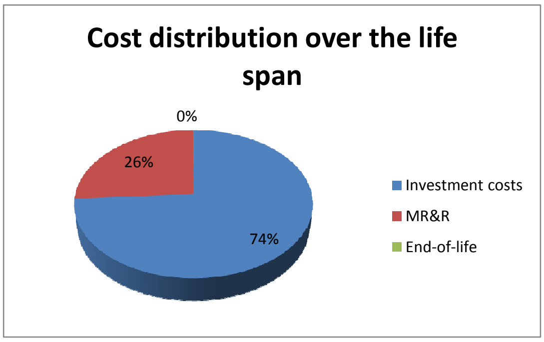
	Agency costs	User costs	Total present cost
Investment costs	17 117 038	8 363	**17 125 401**
MR&R	6 021 686	5 506	**6 027 192**
End-of-life	5 844		**5 844**
Sum	**23 144 568**	**13 869**	**23 158 437**

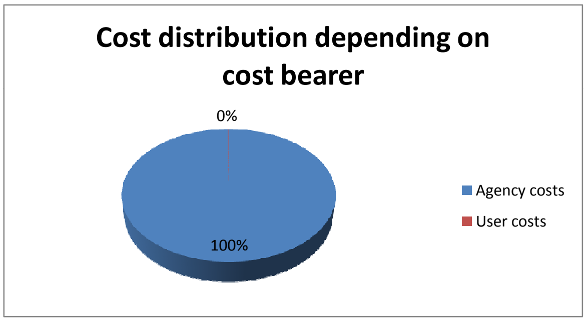




Replacement of bridge

	Agency costs	User costs	Total present cost
Investment costs	17 117 038	24 771	**17 141 809**
MR&R	6 021 686	5 506	**6 027 192**
End-of-life	5 844	0	**5 844**
Sum	**23 144 568**	**30 277**	**22 334 845**





Life cycle cost analysis

General conditions

Name of project:	Steel Sandwich		
Date:	2013-05-02		
Service life	years	100	
Real discount rate	%	3,5%	
ADT on bridge	veh/day	19 715	
Percentage of trucks on bridge	%	5,1%	
Passenger cars on bridge	veh/day	18 710	
Trucks on bridge	veh/day	1 005	
Allowed speed on bridge	km/h	50	
Reduced speed on bridge	km/h		
ADT under bridge	veh/day	87 120	
Percentage of trucks under bridge	%	8,2%	
Passenger cars under bridge	veh/day	79 976	
Trucks under bridge	veh/day	7 144	
Allowed speed under bridge	km/h	70	
Reduced speed under bridge	km/h		
Percentage night traffic, pc	%	11%	
Percentage night traffic, trucks	%	16%	
Hourly cost, car	SEK/h	167	
Hourly cost, truck	SEK/h	347	
Bridge length	m	88	
Bridge width	m	10	
Effective bridge width	m	10	
Bridge area	m^2	880	
Area of surfacing	m^2	880	
Painted area	m^2	2 399	incl. Cross beams
Length of edge beams	m	176	

Investment costs

	Unit price		
Steel	24 500	SEK/ton	
Steel IPE 500	12 700	SEK/ton	Price from Tibnor
Blasting, Primer, Insulation	938	SEK/m^2	Price from Dan Aronsson, DAB
Surfacing	200	SEK/m^2	
Welding	25	SEK/m	Price from Lars-Erik Stridh, ESAB

Welding:	
price per hour	1 950
utilization rate	0,6
welding velocity (m/min)	2,15

Material costs

	steel, IPE 500 [ton]	steel [ton]	blasting & primer [m2]	surfacing [m2]	welding [m]	cost [SEK]
Main beams		69,3				1 697 948
Cross beams (IPE 500)	5,2	6,5				224 350
Bridge deck		137,4	880,0		3 333	4 274 977
Edge beam		41,2				1 010 086
Overlay				880		176 000
Test of air tightness						250 000
					Total cost:	**7 633 361**

User cost new bridge

	construction time [h]	affected pc	affected trucks	reduced speed [km/h]	length of detour [m]	normal way [m]	cost [SEK]
Steel structure northbound	10	4 359	580	50	534	445	1 674
Steel structure southbound	10	4 359	580	40	1 043	616	6 689
						Total cost:	**8 363**

User cost replacement of bridge

	construction time [h]	affected pc	affected trucks	reduced speed [km/h]	length of detour [m]	normal way [m]	cost [SEK]
Steel structure E6 northbound	10	4 359	580	50	534	445	1 674
Steel structure bridge nl 50%	10	1 020	82	40	1 609	1 695	1 052
Steel structure bridge nl 25% Fr	10	510	41	55	2 977	1 695	1 101
Steel structure bridge nl 25% MK	10	510	41	65	4 153	2 837	584
Steel structure southbound	10	4 359	580	40	1 043	616	6 689
Steel structure brdige sl	10	2 039	163	40	1 255	511	3 502
Surfacing nl 50%	8	2 675	144	40	1 609	1 695	1 736
Surfacing nl 25% Fr	8	1 338	72	55	2 977	1 695	2 203
Surfacing nl 25% MK	8	1 338	72	65	4 153	2 837	1 477
Surfacing sl	8	13 359	718	40	1 255	511	17 488
						Total cost:	**37 506**

Average speed normal way [km/h]:	62
Average speed normal way MK [km/h	57

Maintenance & repair

Activity cost

	Activity	Interval [yr]	Cost/unit [SEK/unit]	Quantity	Cost each time	Total present cost
Overlay	Change of insulation [m2]	40	938	880	825 000	261 001
	Change of surfacing [m2]	10	600	880	528 000	1 060 731
Steel	Repainting [m2]	30	1 700	2 399	4 079 116	2 155 554
	Patch painting [m2]	15	1 500	240	359 922	318 645
Disposal	Surfacing	10	-40	224	-8 960	-20 835
					Total cost:	**3 775 097**

Patch painting is assumed to be needed every 15 years, i.e. in the middle between repaintings.

User cost

	Activity	Interval [yr]	disruption time [h]	Affected pc	Affected trucks	reduced speed [k	length of detour [m	normal way [m]	cost [SEK]	Present cost
Overlay	Change of insulation, nl 50%[m2]	40	252	2 675	144	40	1 609	1 695	66 280	20 969
	Change of insulation, nl 25% Fr[m2]	40	252	1 338	72	55	2 977	1 695	69 392	21 953
	Change of insulation, nl 25% MK[m2]	40	252	1 338	72	65	4 153	2 837	36 821	11 649
	Change of insulation,sl [m2]	40	252	13 359	718	40	1 255	511	550 876	174 278
	Change of surfacing, nl 50%[m2]	10	10	292	23	40	1 609	1 695	301	700
	Change of surfacing, nl 25% Fr[m2]	10	10	146	12	55	2 977	1 695	315	732
	Change of surfacing, nl 25% MK[m2]	10	10	146	12	65	4 153	2 837	167	389
	Change of surfacing, sl [m2]	10	10	1 456	117	40	1 255	511	2 500	5 814
Steel	Repainting [m2]	30	20	4 359	580	50	200	200	885	468
									Total cost:	**236 951**

Average speed normal way nl [km/h]: 62
Average speed normal way nl MK [km/h]: 57

End-of-life

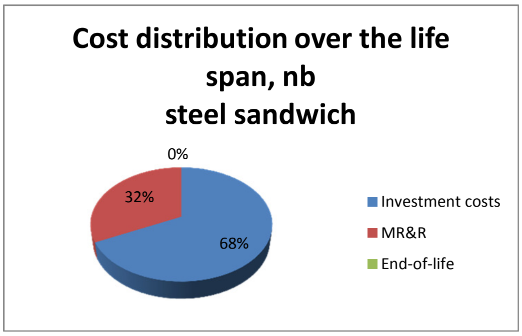
	Cost per unit [SEK]	Quantity [tonne]	Cost [SEK]	Present value [SEK]
Steel	-500	76	-37 888	-1 215
Asphalt	-40	224	-8 960	-287
			Total cost	**-1 502**

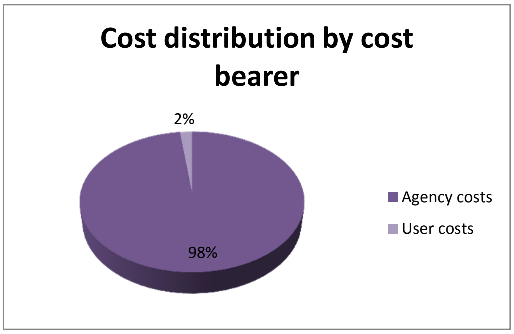
Results

New bridge

	Agency costs	User costs	Total present cost:
Investment costs	8 473 361	8 363	**8 481 724**
MR&R	3 775 097	236 951	**4 012 048**
End-of-life	-1 502		**-1 502**
Sum	**12 246 956**	**245 314**	**12 492 270**

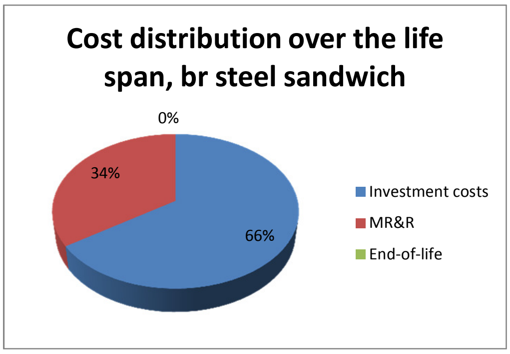
Addition from 24 piles at new construction:
840000

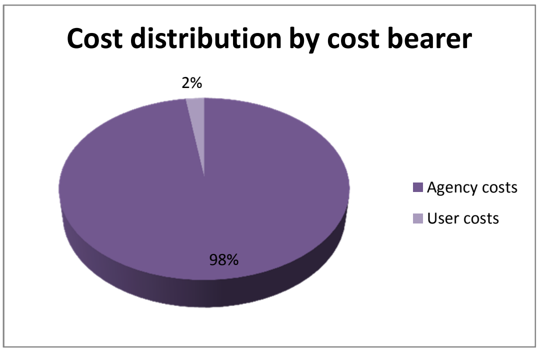




Replacement of bridge

	Agency costs	User costs	Total present cost:
Investment costs	7 633 361	37 506	**7 670 867**
MR&R	3 775 097	236 951	**4 012 048**
End-of-life	-1 502		**-1 502**
Sum	**11 406 956**	**274 457**	**11 681 413**





Appendix I

Maintenance of steel concrete bridges with composite action

In order to make a realistic estimation of the maintenance needed for a conventional steel(concrete composite bridge a number of professionals were consulted. Also, information on the construction of the specific bridge 14-1531-1 at Ullevimotet was gathered.

Tomas Svensson, COWI 2013-03-26

Activity	**Interval**	**Duration**	**Others**
Change of insulation	Approx. 30-40 years	Approx. 2 months	Half the bridge at a time
Replacement of edge beam	Approx. 50 years	Approx. 3 months	One lane closed
Repainting	Approx. 25 years		At night? Affects the traffic below, free height for scaffolding?
Resurfacing, asphalt	Approx. 10-15 years, max 15		

John-Erik Fredriksson, COWI 2013-03-27

Activity	**Interval**	**Duration**	**Others**
Replacement of edge beam	Approx. 40 years	13 w in total	Close 1 lane/edge beam, 30 km/h
Change of insulation, drainage	Approx. 40 years	1,5 w/side	2 traffic lands at a time
Repainting of steel	Approx. 60 years		At night, 1-2 lanes at a time
Resurfacing, asphalt	Approx. 7 years	2 nights	Night, 2 tl at a time
Impregnation and concrete repair			Negligible
Patch painting			At night, probably negligible
Concrete repair, columns, 0-30mm	Approx. 40 years	1 week	At night? A traffic lane will need to be closed

For a new construction, closing of E6 probably will be needed for lifting of the steel beams only. Possibly for painting of the beams as well. Otherwise, formwork etc. is prepared before lifting so that no closing of E6 is needed for curing and finishing.

Daniel Rönnebjerg, COWI 2013-04-02

Activity	**Interval**	**Duration**	**Others**
Repainting	Approx. 20-25år		

Per Thunstedt, Trafikverket 2013-04-04

Activity	**Interval**	**Duration**	**Others**
Replacement of edge beam	Approx. 80 years		
Change of insulation, expansion joints	Approx. 40 years	2-3weeks/side	1 direction at a time
Repainting, steel	Approx. 35-40 years		At night, 1-2traffic lanes at a time
Cleaning of edge beams	1 year		1 tl, at night
Cleaning of expansion joints	6 months		
Cleaning of main beams	1year		Should probably be done, but isn't today
Resurfacing, asphalt	Approx. 10 years	A couple of nights, probably 1 week	Night, 2 lanes at a time
Impregnation of edge beam/columns	Approx. 10 years		This demand has just been removed
Repair of surfacing	2-3ggr/exchange	Some square meter	Negligible
Concrete repair on columns/edge beams, 0-30mm			Should not be needed

On demolition: The concrete is crushed using machines at the site. The reinforcement stays in place and can then be recycled. The crushed material can be used for road construction or something similar. Could probably be done during one weekend, running 24 hours every day. Would probably cost SEK2-3 million (including traffic).

A tip: There is an investigation on corrosivity and painting on composite action bridges by Patrik Reuterswärd.
Could contact: Jan-Olof Schröder who was construction manager at Trafikverket while building the Ullevi bridge.

Maintenance:

- Cleaning of edge beam every year, close one traffic lane at night
- Clean expansion joints 2 times/year
- The surfacing was changed in the summer of 2012 (which was on overtime). Generally it needs to be changed every 10 years. Then, change of drainage is included. This takes a couple of night (approximately 1 week).
- Insulation is changed every 40 years. The traffic is closed one side at a time, expansion joints are changed at the same time. Approx. 3 weeks/direction. But let's say that it will take totally 4 weeks for this bridge (quite small).
- Repainting: Should be cleaned 1 time/year. The Älvsborg bridge was built in 1966. In 1994 it was repainted (i.e after 28 years) and now it needs to be repainted again, the work is scheduled to 2018. The Älvsborg bridge, though, is located in a very difficult environment. The bridges in Ringömotet (code 658-1 in BaTMan), where the environment is more friendly, were built approx. 1968 and were repainted in 2008. There is an investigation on painting by Patrik Reuterswärd.

Jan-Olof Schröder, Göteborgs byggledning 2013-04-04
The construction of the bridge in Ullevimotet (14-1531-1) began in the autumn of 1994 and was finished before the Athletics world cup in '95. PEAB was the constructor. In the building process the beams were lifted in place. Then security scaffolding was placed at the lower flange of the I-girders.

Generally, free height under bridges should be at least 4.5 meters. Transport of materials can be maximum 5 m wide.

In this case, the traffic will be disrupted during lifting of the beams and piling for the mid support only. That would probably take two nights. When the bridge was built it replaced a bridge in the same location, just beside it. This old bridge was kept until the new was finished so in this case the traffic did not have to be rerouted. The demolition was made during the weekend/holiday around May 1st. The demolition was made one side at a time. It probably cost SEK5-10,000/h.

Christer Andersson was responsible for the construction at PEAB.

Appendix J
Emission vectors from openLCA

These emission vectors for FRP, polymer concrete based on polyester and polymer concrete based on epoxy resin were obtained from openLCA using the methods ReCiPe (H) and USEtox. They correspond to the impact categories and methods considered in BridgeLCA where they will be implemented.

			Emission vectors		
Impact category	Method	Unit	FRP [1 kg]	Polyester PC [1 kg]	Epoxy PC [1 kg]
GWP	ReCiPe (H)	kg CO2 eq	3,62E+00	1,49E+00	9,08E-01
ODP	ReCiPe (H)	kg CFC-11 eq	3,40E-07	1,54E-07	9,33E-09
EP	ReCiPe (H)	kg P eq	1,16E-03	3,40E-04	2,82E-05
AP	ReCiPe (H)	kg SO2 eq	1,49E-02	3,42E-03	5,22E-03
FD	ReCiPe (H)	kg oil eq	1,21E+00	5,10E-01	3,89E-01
ET	USEtox	CTUe	1,52E+00	4,97E-01	3,23E-01
HTC	USEtox	CTUh	1,68E-07	4,26E-08	5,11E-08
HTNC	USEtox	CTUh	7,21E-07	1,11E-07	1,16E-07

GWP-global warming potential
ODP-ozone depletion potential
EP-freshwater eutrophication
AP-terrestrial acidification
FD-fossil depletion
ET-ecotoxicity
HTC-human toxicity cancer
HTNC-human toxicity non-cancer

To make sure that the impact vectors obtained form openLCA are compatible with the impact vectors in BridgeLCA a verification was made for the material "Concrete, at plant". As can be seen the values for the impact categories assessed with the *ReCiPe (H), midpoint* method are identical while the values obtained with the *USEtox method* show large differences.

			Concrete, at plant [1 m3]	
Impact category	Method	Unit	openLCA	BridgeLCA
GWP	ReCiPe (H)	kg CO2 eq	2,61E+02	2,61E+02
ODP	ReCiPe (H)	kg CFC-11 eq	8,84E-06	8,84E-06
EP	ReCiPe (H)	kg P eq	4,44E-01	4,44E-01
AP	ReCiPe (H)	kg SO2 eq	1,37E-02	1,37E-02
FD	ReCiPe (H)	kg oil eq	2,57E+01	2,57E+01
ET	USEtox	CTUe	2,70E+01	8,43E-02
HTC	USEtox	CTUh	3,17E-06	1,11E-08
HTNC	USEtox	CTUh	1,19E-05	1,18E-08

Appendix K
Result of LCA, midpoint level and normalized

Midpoint results

Scenario 1 -new bridge

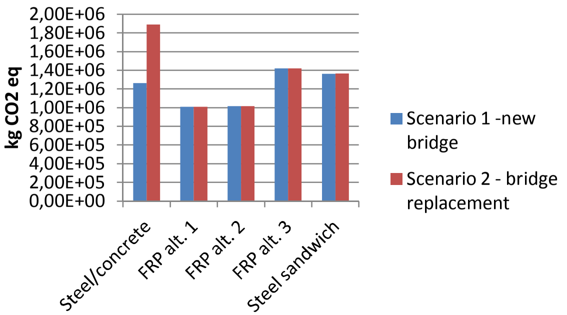
	Category equivalent	Steel/concrete	FRP alt. 1	FRP alt. 2	FRP alt. 3	Steel sandwich
Global warming potential	[kg CO2 eq]	1,26E+06	1,01E+06	1,02E+06	1,42E+06	1,36E+06
Ozone depletion	[kg CFC-11 eq]	1,36E-01	8,14E-02	8,17E-02	1,15E-01	1,43E-01
Terrestrial acidification	[kg SO2 eq]	3,64E+03	3,24E+03	3,26E+03	4,86E+03	4,22E+03
Freshwater eutrophication	[kg P eq]	2,20E+02	3,38E+02	3,43E+02	4,93E+02	3,42E+02
Fossil depletion	[kg oil eq]	4,36E+05	3,39E+05	3,41E+05	4,76E+05	5,26E+05
Human toxicity, cancer	[CTUh]	1,06E-04	2,75E-02	2,75E-02	4,27E-02	1,28E-04
Human toxicity, non-cancer	[CTUh]	2,72E-05	9,88E-02	9,88E-02	1,65E-01	2,82E-05
Ecotoxicity	[CTUe]	1,50E+03	2,79E+05	2,79E+05	4,18E+05	1,63E+03

Scenario 2 - bridge replacement

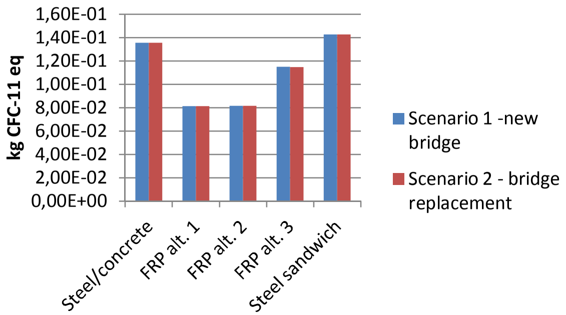
	Category equivalent	Steel/concrete	FRP alt. 1	FRP alt. 2	FRP alt. 3	Steel sandwich
Global warming potential	[kg CO2 eq]	1,89E+06	1,01E+06	1,02E+06	1,42E+06	1,36E+06
Ozone depletion	[kg CFC-11 eq]	1,35E-01	8,13E-02	8,15E-02	1,15E-01	1,43E-01
Terrestrial acidification	[kg SO2 eq]	5,07E+03	3,24E+03	3,27E+03	4,86E+03	4,23E+03
Freshwater eutrophication	[kg P eq]	2,20E+02	3,38E+02	3,43E+02	4,93E+02	3,42E+02
Fossil depletion	[kg oil eq]	4,36E+05	3,38E+05	3,41E+05	4,75E+05	5,25E+05
Human toxicity, cancer	[CTUh]	1,06E-04	2,75E-02	2,75E-02	4,27E-02	1,28E-04
Human toxicity, non-cancer	[CTUh]	2,70E-05	9,88E-02	9,88E-02	1,65E-01	2,80E-05
Ecotoxicity	[CTUe]	1,50E+03	2,79E+05	2,79E+05	4,18E+05	1,63E+03

Graphic midpoint results:

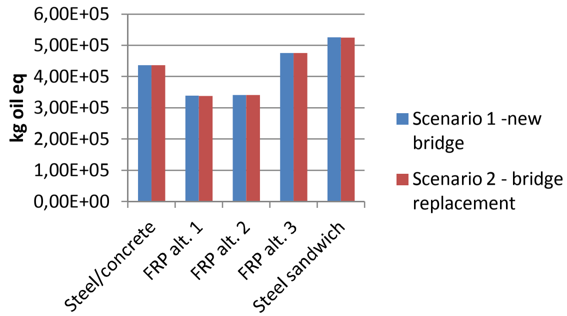
Global warming potential - midpoint results



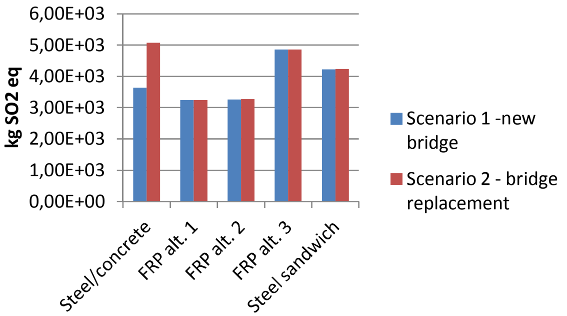
Ozone depletion potential - midpoint results



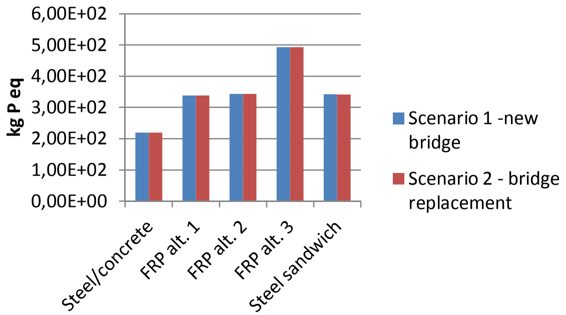
Fossil depletion - midpoint results



Terrestrial acidification- midpoint results



Freshwater eutrophication - midpoint results



Normalized results

Scenario 1

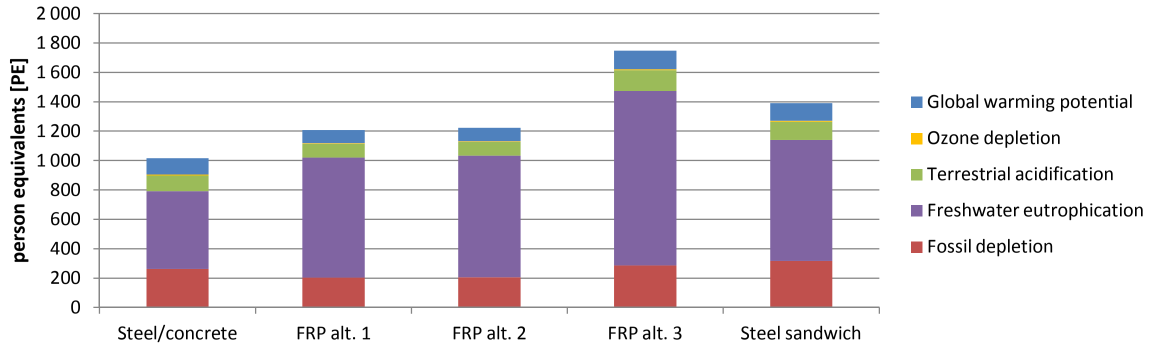
	Steel/concrete	FRP alt. 1	FRP alt. 2	FRP alt. 3	Steel sandwich
Global warming potential	1,13E+02	9,00E+01	9,07E+01	1,27E+02	1,22E+02
Ozone depletion	6,17E+00	3,70E+00	3,72E+00	5,23E+00	6,50E+00
Terrestrial acidification	1,06E+02	9,42E+01	9,50E+01	1,41E+02	1,23E+02
Freshwater eutrophication	5,30E+02	8,16E+02	8,28E+02	1,19E+03	8,25E+02
Fossil depletion	2,62E+02	2,04E+02	2,05E+02	2,86E+02	3,16E+02
Sum [PE]	**1017**	**1207**	**1222**	**1748**	**1391**

Scenario 2

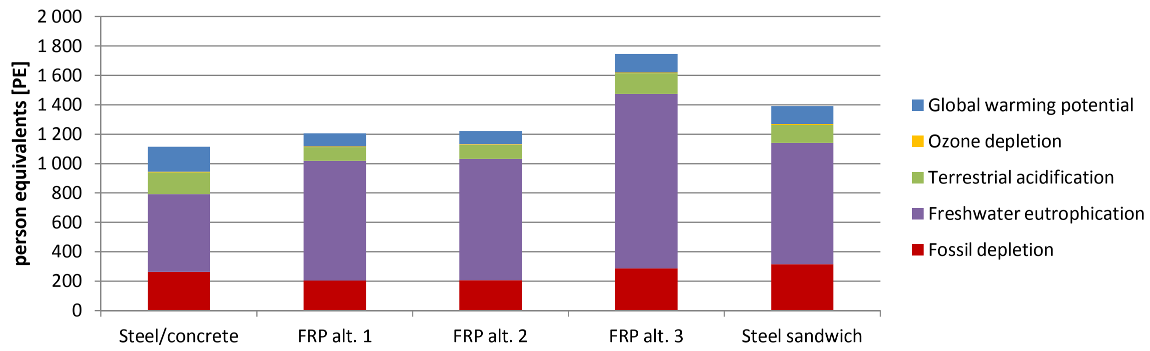
	Steel/concrete	FRP alt. 1	FRP alt. 2	FRP alt. 3	Steel sandwich
Global warming potential	1,68E+02	9,00E+01	9,07E+01	1,27E+02	1,22E+02
Ozone depletion	6,16E+00	3,70E+00	3,71E+00	5,22E+00	6,49E+00
Terrestrial acidification	1,48E+02	9,42E+01	9,50E+01	1,41E+02	1,23E+02
Freshwater eutrophication	5,29E+02	8,15E+02	8,27E+02	1,19E+03	8,24E+02
Fossil depletion	2,62E+02	2,03E+02	2,05E+02	2,86E+02	3,16E+02
Sum [PE]	**1114**	**1207**	**1222**	**1747**	**1391**

Graphic normalized results

Comparison of normalized LCIA results - scenario 1



Comparison of normalized LCIA results - scenario 2



Weighting factors	FD	AP	EP	GWP	ODP
US-EPA	5	5	5	16	5
Harvard	7	9	9	11	11
BEES default	9	9	9	9	8
EDIP	0	1,3	1,2	1,3	23

Weighted results scenario 1

US-EPA

	Steel/concrete	FRP alt. 1	FRP alt. 2	FRP alt. 3	Steel sandwich
Global warming potential	1,80E+03	1,44E+03	1,45E+03	2,03E+03	1,94E+03
Ozone depletion	3,08E+01	1,85E+01	1,86E+01	2,62E+01	3,25E+01
Terrestrial acidification	5,30E+02	4,71E+02	4,75E+02	7,07E+02	6,14E+02
Freshwater eutrophication	2,65E+03	4,08E+03	4,14E+03	5,94E+03	4,12E+03
Fossil depletion	1,31E+03	1,02E+03	1,03E+03	1,43E+03	1,58E+03
SUM	**6,32E+03**	**7,03E+03**	**7,11E+03**	**1,01E+04**	**8,29E+03**

Harvard

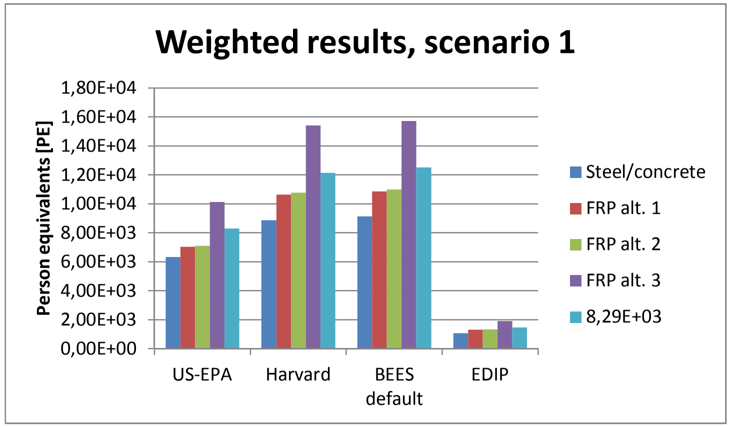
	Steel/concrete	FRP alt. 1	FRP alt. 2	FRP alt. 3	Steel sandwich
Global warming potential	1,24E+03	9,90E+02	9,98E+02	1,39E+03	1,34E+03
Ozone depletion	6,78E+01	4,07E+01	4,09E+01	5,76E+01	7,15E+01
Terrestrial acidification	9,53E+02	8,48E+02	8,55E+02	1,27E+03	1,11E+03
Freshwater eutrophication	4,77E+03	7,34E+03	7,45E+03	1,07E+04	7,42E+03
Fossil depletion	1,84E+03	1,43E+03	1,44E+03	2,00E+03	2,21E+03
SUM	**8,86E+03**	**1,06E+04**	**1,08E+04**	**1,54E+04**	**1,21E+04**

BEES default

	Steel/concrete	FRP alt. 1	FRP alt. 2	FRP alt. 3	Steel sandwich
Global warming potential	1,01E+03	8,10E+02	8,16E+02	1,14E+03	1,09E+03
Ozone depletion	4,93E+01	2,96E+01	2,97E+01	4,19E+01	5,20E+01
Terrestrial acidification	9,53E+02	8,48E+02	8,55E+02	1,27E+03	1,11E+03
Freshwater eutrophication	4,77E+03	7,34E+03	7,45E+03	1,07E+04	7,42E+03
Fossil depletion	2,36E+03	1,83E+03	1,85E+03	2,57E+03	2,84E+03
SUM	**9,15E+03**	**1,09E+04**	**1,10E+04**	**1,57E+04**	**1,25E+04**

EDIP

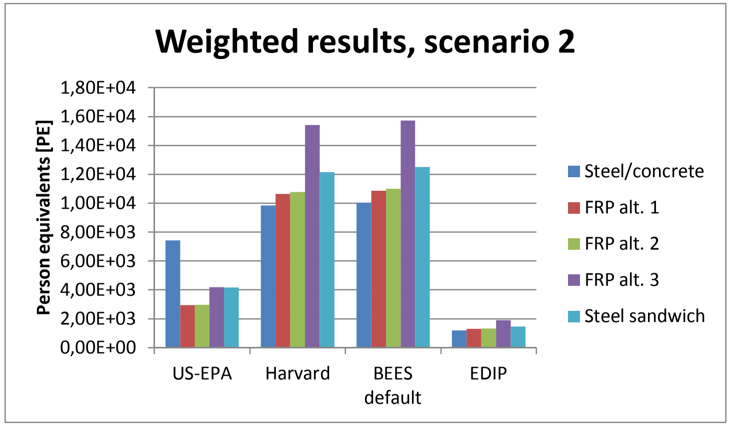
	Steel/concrete	FRP alt. 1	FRP alt. 2	FRP alt. 3	Steel sandwich
Global warming potential	1,46E+02	1,17E+02	1,18E+02	1,65E+02	1,58E+02
Ozone depletion	1,42E+02	8,52E+01	8,55E+01	1,20E+02	1,49E+02
Terrestrial acidification	1,38E+02	1,22E+02	1,23E+02	1,84E+02	1,60E+02
Freshwater eutrophication	6,36E+02	9,79E+02	9,93E+02	1,43E+03	9,89E+02
Fossil depletion	0,00E+00	0,00E+00	0,00E+00	0,00E+00	0,00E+00
SUM	**1,06E+03**	**1,30E+03**	**1,32E+03**	**1,89E+03**	**1,46E+03**



Weighted results scenario 2

US-EPA

	Steel/concrete	FRP alt. 1	FRP alt. 2	FRP alt. 3	Steel sandwich
Global warming potential	2,69E+03	1,44E+03	1,45E+03	2,03E+03	1,95E+03
Ozone depletion	3,08E+01	1,85E+01	1,85E+01	2,61E+01	3,24E+01
Terrestrial acidification	7,38E+02	4,71E+02	4,75E+02	7,07E+02	6,15E+02
Freshwater eutrophication	2,65E+03	0,00E+00	0,00E+00	0,00E+00	0,00E+00
Fossil depletion	1,31E+03	1,02E+03	1,02E+03	1,43E+03	1,58E+03
SUM	**7,42E+03**	**2,95E+03**	**2,97E+03**	**4,19E+03**	**4,17E+03**



Harvard

	Steel/concrete	FRP alt. 1	FRP alt. 2	FRP alt. 3	Steel sandwich
Global warming potential	1,85E+03	9,90E+02	9,98E+02	1,39E+03	1,34E+03
Ozone depletion	6,78E+01	4,07E+01	4,08E+01	5,75E+01	7,14E+01
Terrestrial acidification	1,33E+03	8,48E+02	8,55E+02	1,27E+03	1,11E+03
Freshwater eutrophication	4,77E+03	7,34E+03	7,45E+03	1,07E+04	7,42E+03
Fossil depletion	1,83E+03	1,42E+03	1,43E+03	2,00E+03	2,21E+03
SUM	**9,85E+03**	**1,06E+04**	**1,08E+04**	**1,54E+04**	**1,21E+04**

BEES default

	Steel/concrete	FRP alt. 1	FRP alt. 2	FRP alt. 3	Steel sandwich
Global warming potential	1,52E+03	8,10E+02	8,17E+02	1,14E+03	1,10E+03
Ozone depletion	4,93E+01	2,96E+01	2,97E+01	4,18E+01	5,19E+01
Terrestrial acidification	1,33E+03	8,48E+02	8,55E+02	1,27E+03	1,11E+03
Freshwater eutrophication	4,77E+03	7,34E+03	7,45E+03	1,07E+04	7,42E+03
Fossil depletion	2,36E+03	1,83E+03	1,84E+03	2,57E+03	2,84E+03
SUM	**1,00E+04**	**1,09E+04**	**1,10E+04**	**1,57E+04**	**1,25E+04**

EDIP

	Steel/concrete	FRP alt. 1	FRP alt. 2	FRP alt. 3	Steel sandwich
Global warming potential	2,19E+02	1,17E+02	1,18E+02	1,65E+02	1,58E+02
Ozone depletion	1,42E+02	8,50E+01	8,53E+01	1,20E+02	1,49E+02
Terrestrial acidification	1,92E+02	1,23E+02	1,24E+02	1,84E+02	1,60E+02
Freshwater eutrophication	6,35E+02	9,79E+02	9,93E+02	1,43E+03	9,89E+02
Fossil depletion	0,00E+00	0,00E+00	0,00E+00	0,00E+00	0,00E+00
SUM	**1,19E+03**	**1,30E+03**	**1,32E+03**	**1,89E+03**	**1,46E+03**

Appendix L

Design of bridge with steel sandwich deck

Dimensions

$L_b := 21.95\text{m}$ Span length

Main girders

$h_w := 1300\text{mm}$

$t_w := 12\text{mm}$

$b_{bf} := 700\text{mm}$

$t_{bf} := 50\text{mm}$

$n_{bf} := 2$

$n_w := 2$

Top flange (deck plate)

$b_{tf} := 10\text{m}$

$t_{tf} := 12\text{mm}$

Core

$t_c := 6\text{mm}$

$b_c := 50\text{mm} + 2\cdot 122\text{mm} + 110\text{mm} = 0.404\,\text{m}$

$n_c := 73$

Edge beams

The edge beams in the FEM-model was designed as a box girder. To optimize the solution, a C-profile is desired.

Moment of inertia of the edge beam created in the FEM model (box section):

$$I_{eb} := \frac{(600\text{mm})^3\cdot 25\text{mm}\cdot 2}{12} + \frac{(25\text{mm})^3\cdot 125\text{mm}\cdot 2}{12} + 2\cdot 25\text{mm}\cdot 125\text{mm}\cdot\left(\frac{600\text{mm}}{2} + \frac{25\text{mm}}{2}\right)^2 = 1.511\times 10^9$$

The same moment of inertia is needed for an edge beam with C-profile:

$$I_{eb2} := \frac{(600mm)^3 \cdot 35mm}{12} + \frac{(35mm)^3 \cdot 130mm \cdot 2}{12} + 2 \cdot 35mm \cdot 130mm \cdot \left(\frac{600mm}{2} + \frac{35mm}{2}\right)^2 = 1.548 \times 10^9 \cdot$$

Choose the dimensions used in I_{eb2} above

$t_{ebf} := 35mm$ $t_{ebw} := 35mm$

$b_{ebf} := 130mm$ $h_{ebw} := 600mm$

$n_{eb} := 2$

Cross beams mid (IPE 500)

$h_{cbm} := 500mm$ $b_{cbm} := 200mm$

$d_{cbm} := 10.2mm$ $t_{cbm} := 16mm$

$L_{cbm} := 5m$

$n_{cbm} := 4$

Cross beams edge (IPE 500, modified)

$h_{cbe1} := 500mm$ $b_{cbe} := 200mm$

$h_{cbe2} := 276mm$

$d_{cbe} := 10.2mm$ $t_{cbe} := 16mm$

$L_{cbe1} := 2.5m$

$L_{cbe2} := 2.51m$

$n_{cbe} := 4 \cdot 2 = 8$

Mass of 1 span in 1 bridge

$$\begin{aligned} V_{bridge} := & \left[h_w \cdot t_w \cdot n_w + b_{bf} \cdot t_{bf} \cdot n_{bf} + t_{tf} \cdot b_{tf} + \left(t_{ebf} \cdot 2b_{ebf} + t_{ebw} \cdot h_{ebw}\right) \cdot n_{eb}\right] \cdot L_b \ldots = 8.32 \cdot m^3 \\ & + t_c \cdot b_c \cdot n_c \cdot b_{tf} + \left(h_{cbm} \cdot d_{cbm} + b_{cbm} \cdot t_{cbm}\right) \cdot L_{cbm} \cdot n_{cbm} \ldots \\ & + \left[\left(\frac{h_{cbe1} + h_{cbe2}}{2} \cdot d_{cbe} + b_{cbe} \cdot t_{cbe}\right) \cdot L_{cbe1} + b_{cbe} \cdot t_{cbe} \cdot L_{cbe2}\right] \cdot n_{cbe} \end{aligned}$$

$$\rho_{steel} := 7800\,\frac{kg}{m^3}$$

$$m_{gird} := \left(h_w \cdot t_w \cdot n_w + b_{bf} \cdot t_{bf} \cdot n_{bf}\right) \cdot L_b \cdot \rho_{steel} = 17.326 \cdot tonne$$

$$m_{cbm} := \left(h_{cbm} \cdot d_{cbm} + b_{cbm} \cdot t_{cbm}\right) \cdot L_{cbm} \cdot n_{cbm} \cdot \rho_{steel} = 1.295 \cdot tonne$$

$$m_{cbe} := \left[\left(\frac{h_{cbe1} + h_{cbe2}}{2} \cdot d_{cbe} + b_{cbe} \cdot t_{cbe}\right) \cdot L_{cbe1} + b_{cbe} \cdot t_{cbe} \cdot L_{cbe2}\right] \cdot n_{cbe} \cdot \rho_{steel} = 1.618 \cdot tonne$$

$$m_{deck} := \left(t_{tf} \cdot b_{tf} \cdot L_b + t_c \cdot b_c \cdot n_c \cdot b_{tf}\right) \cdot \rho_{steel} = 34.347 \cdot tonne$$

$$m_{eb} := \left(t_{ebf} \cdot 2b_{ebf} + t_{ebw} \cdot h_{ebw}\right) \cdot n_{eb} \cdot L_b \cdot \rho_{steel} = 10.307 \cdot tonne$$

$$m_{steel} := \rho_{steel} \cdot V_{bridge} = 64.893 \cdot tonne$$

$$m_{steel} := m_{gird} + m_{cbm} + m_{cbe} + m_{deck} + m_{eb} = 64.893 \cdot tonne$$

whole bridge:

$$m_{steel} \cdot 4 = 259.573 \cdot tonne$$

$$t_{asp} := 0.07m$$

$$\rho_{asp} := 2.38\,\frac{tonne}{m^3}$$

$$m_{asp} := L_b \cdot b_{tf} \cdot t_{asp} \cdot \rho_{asp} = 36.569 \cdot tonne$$

$$m_{tot} := m_{asp} + m_{steel} = 101.462 \cdot tonne$$

whole bridge:

$$m_{tot} \cdot 4 = 405.848 \cdot tonne$$

Dead load

$$q_{self} := \frac{m_{tot} \cdot g}{L_b \cdot b_{tf}} = 4.533 \times 10^{-3} \cdot MPa$$

Painted area

$$A_p := \left(2 \cdot h_w \cdot n_w + 2 \cdot b_{bf} \cdot n_{bf}\right) \cdot L_b + b_c \cdot n_c \cdot b_{tf} + \left(3 \cdot b_{ebf} + 2 \cdot h_{ebw}\right) \cdot L_b \cdot n_{eb} \ldots = 599.873\, m^2$$
$$+ \left(2 \cdot h_{cbm} + 3 \cdot b_{cbm}\right) \cdot L_{cbm} \cdot n_{cbm} \ldots$$
$$+ \left[\left(2 \cdot \frac{h_{cbe1} + h_{cbe2}}{2} + b_{cbe}\right) \cdot L_{cbe1} + 2 \cdot b_{cbe} \cdot L_{cbe2}\right] \cdot n_{cbe}$$

whole bridge:

$$A_p \cdot 4 = 2.399 \times 10^3\, m^2$$

Load bearing capacity

ULS:

$\sigma_{max} := 180.048 MPa$ Maximum tensile stress in the steel girders

$f_y := 355 MPa$

$\gamma_{steel} := 1.0$

$\frac{\sigma_{max}}{f_y} \cdot \gamma_{steel} = 0.507$ **OK**

SLS:

$d_{max} := 34.26 mm$ Maximum deflection in the steel girders

$d_{lim} := \frac{L_b}{400} = 0.055\,m$ Deflection limit

$\frac{d_{max}}{d_{lim}} = 0.624$ **OK**

FLS:

$\sigma_{maxFLS} := 120.032 MPa$ Maximum stress at the weld

$\sigma_{minFLS} := 28.46 MPa$ Minimum stress at the weld (caused by dead load only)

$\Delta\sigma_{Ed} := \sigma_{maxFLS} - \sigma_{minFLS} = 91.572 \cdot MPa$

$\Delta\sigma_{Rd} := 100 MPa$ Allowed stress range at the weld

$\frac{\Delta\sigma_{Ed}}{\Delta\sigma_{Rd}} = 0.916$ **OK**

Appendix M
Summary of data for all alternatives and rough calculations on substructure

Self weights for all bridges:

Steel/concrete alternative

$m_{steel} := 118.265 \text{tonne}$

$m_{asp} := 224 \text{tonne}$

$\rho_{con} := 2500 \dfrac{kg}{m^3}$

$t_{con} := 0.25m$

$A_{bridge} := 20m \cdot 44m = 880\, m^2$

$m_{con} := \rho_{con} \cdot t_{con} \cdot A_{bridge} = 550 \cdot \text{tonne}$

$m_{reinf} := 36 \text{tonne}$

$m_{ub} := m_{steel} + m_{asp} + m_{con} + m_{reinf} = 928.265 \cdot \text{tonne}$ total weight of the steel/concrete concept

$A_{paintu} := 918.505 m^2$

$R_{supu} := 4050.7 kN$ per beam/support

Self-weight mid support:

$n_{sup} := 4$

$d_{sup} := 1m$ $r_{sup} := \dfrac{d_{sup}}{2} = 0.5\, m$

$h_{sup} := (440 + 1450 \cdot 3)mm = 4.79\, m$

$V_{sup} := n_{sup} \cdot r_{sup}^{\;2} \cdot \pi \cdot h_{sup} = 15.048 \cdot m^3$

$m_{sup} := V_{sup} \cdot \rho_{con} = 37.621 \cdot \text{tonne}$

$R_s := m_{sup} \cdot g = 368.932 \cdot kN$

Alternativ 1:

$m_{st1} := 121.446 tonne$

$\rho_{pc} := 2260 \frac{kg}{m^3}$

$m_{pc} := \rho_{pc} \cdot 30mm \cdot A_{bridge} = 59.664 \cdot tonne$

$m_{FRP} := 103.69 \frac{kg}{m^2} \cdot A_{bridge} = 91.247 \cdot tonne$

$m_{alt1} := m_{st1} + m_{pc} + m_{FRP} = 272.357 \cdot tonne$

$A_{paint1} := 1222m^2$

$R_{alt1} := 1678.6kN$ reaction force to mid support from worst loaded beam

Alternativ 2:

$m_{st2} := (125.86) tonne = 125.86 \cdot tonne$

$m_{alt2} := m_{st2} + m_{pc} + m_{FRP} = 276.771 \cdot tonne$

$A_{paint} := 1221m^2$

$R_{alt2} := 3107kN$ per beam

Alternativ 3:

$m_{st3} := 153.3tonne + 11.8tonne = 165.1 \cdot tonne$

$m_{FRP2} := 2m_{FRP} = 182.494 \cdot tonne$

$m_{alt3} := m_{st3} + m_{FRP2} + m_{pc} = 407.258 \cdot tonne$

$A_{paint3} := 1184m^2$

$R_{alt3} := 2964.7kN$ per beam

Steel sandwich bridge

$m_{stSS} := 259.573\,tonne$

$m_{aspSS} := 146.275\,tonne$

$m_{SS} := m_{stSS} + m_{aspSS} = 405.848 \cdot tonne$

$A_{paintSS} := 2399m^2$

Reaction force at mid support is unknown

Total reaction forces on ground at the mid support:

Ullevi bridge

$R_u := R_{supu} \cdot 4 + R_s = 16.572 \cdot MN$

Alt 1

$R_1 := R_{alt1} \cdot 7 + R_s = 12.119 \cdot MN$

Alt 2

$R_2 := R_{alt2} \cdot 4 + R_s = 12.797 \cdot MN$

Alt 3

$R_3 := R_{alt3} \cdot 5 + R_s = 15.192 \cdot MN$

$\frac{R_1}{R_u} = 73.131 \cdot \%$ $\frac{R_2}{R_u} = 77.221 \cdot \%$ $\frac{R_3}{R_u} = 91.677 \cdot \%$

The group of piles placed under the original Ullevi bridge is designed to carry a compressive load o 18.7MN:

$p_{max} := 778kN$

$n_{piles} := 24$

$l_{pmean} := 7m$

$p_{tot} := p_{max} \cdot n_{piles} = 18.672 \cdot MN$

Based on this we assume that the total reaction forces presented above are reasonable. To make a rough estimation of the need for piles:

The total reaction force for alternative 1 and 2 is about 75% of the total reaction force for the Ullevi bridge. Therefore we conclude that it might be possible to reduce the number of piles with 25 % i.e from 24 to 18. However, the piles are pced in fur groups of six. A reduction of six piles could not be evenly ditributed on these four groups. Therefore the number is reduced to 20 piles to be on the safe side.

For alternative 3 the total reaction force is 92% of the one in the Ullevi bridge, no reduction of piles is made.

There is no risk of tension in the piles below the mid support according to original design, there should not be for the newer ligther designs either since the bridge is continuous over the support.