European Commission

Sustainable steel-composite bridges in built environment

(SBRI)

Research and Innovation

EUR 26322 EN

EUROPEAN COMMISSION
Directorate-General for Research and Innovation
Directorate G — Industrial Technologies
Unit G.5 — Research Fund for Coal and Steel

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European Commission

Research Fund for Coal and Steel

Sustainable steel-composite bridges in built environment

(SBRI)

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Grant Agreement RFSR-CT-2009-00020
1 July 2009 to 30 June 2012

Final report

Directorate-General for Research and Innovation

2013 EUR 26322 EN

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Luxembourg: Publications Office of the European Union, 2013

ISBN 978-92-79-34586-9
doi:10.2777/50286

Printed in Luxembourg

PRINTED ON WHITE CHLORINE-FREE PAPER

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FINAL SUMMARY

Objectives of the project

The overall objective of this project is to promote steel in the bridge construction market by pointing out benefits of steel-composite bridges regarding sustainability by means of an integral holistic approach combining Life-cycle Assessment (LCA), Life-cycle Costs (LCC) and Life-cycle Performance (LCP) analyses.

Traditionally bridges are designed to achieve minimal initial costs leading to a wide spread domination of concrete bridges. In regard of sustainability not only the construction stage but the entire life-cycle of 100 years must be taken into account. The approach was applied to realistic case studies in order to elaborate advantages and influences.

The project aim was first to collect extensive data regarding LCC, LCA and LCP for all the life-cycle stages. As bridges are designed to cover a lifespan of more than 100 years inspections and maintenance actions need to be given a special focus. The database forms then the bases for the detailed investigations on LCC, LCA and LCP. Special focus is given to the Life-cycle Performance in regard of different degradation processes in composite bridges. Suitable models to be integrated in the analysis are to be provided for typical effects with the help of prototype testing. The analysis of complete case studies aims at possible comparisons and improvements by variations. The development of a tool, a handbook and the organisation of a workshop is to assist the application and dissemination process of the results.

Description of activities and discussion

During the project term investigations were carried out and presented to the project partners at six coordination meetings (plus one meeting at the end of 2012) for discussion and coordination. Intermediate results were reported regularly in the technical reports, [37], [38], [39], [42]. Especially the selection of the analysed case studies were discussed widely and continuously as variants appeared to be of interest.

Gathering information and data forms an essential basis for the work done in the successive work packages. First, the selection of the bridge types and the corresponding functionality (crossing or motorway bridge) and second, the definition of variants and optimizations to be analysed, were fundamental. Life-cycle analyses were carried out on the bridge design of these representative European bridge situations. Thereby a databank was created, integrated and used by the SBRI-tool, the relevance of which goes far beyond the single aims of this project but may form the basis of a future sustainable bridge design across Europe.

Besides, special feature inspection and maintenance strategies have been collected by the partners involved coming from 6 different countries and including a number of representatives of bridge owners and authorities. These different strategies were condensed to a “Standard European Strategy” which formed the basis for the evaluation of the whole life-cycle analyses. As the operation phase gives a very important input to the outcome of the overall LCA and LCC analyses, this standard strategy is a crucial point. Therefore two additional scenarios were also looked at throughout the project: the scenario “lack of money” and the scenario”prolonging the lifetime of a bridge”.

All potential environmental impacts of bridges due to their construction (including also the material production), operation and maintenance over their service life and final demolition were calculated. Adaptations for life-cycle environmental analyses, existing mainly in the building sectors, were made for the specific needs of bridge structures. Eight impact categories were not only analysed during the stages of the life-cycle but also the contribution of main processes to the impact categories was identified. Variants of the case studies allow for comparisons and conclusions regarding different environmental performance.

The methodology for LCC analysis of steel-composite bridges was developed including especially also user costs. Main life-cycle costs were identified in order to assess the cost-effectiveness of design solutions. The life-cycle costs were investigated not only for the construction stage but also for the operation and till the end, covering the entire life-cycle. The application to the case studies showed the importance of the year in which the costs accrue, as a yearly discount-rate of money considered.

As various degradation processes affect bridges during their lifetime and influence the sequence of inspection and maintenance, three typical ones found in steel-composite bridges, namely fatigue, corrosion and carbonation, were analysed herein. As different methods were needed it was decided to approach these processes separately and with the help of prototype testing, models for the life-cycle performance of critical details were checked and elaborated. Experimental investigations on the fatigue behaviour of transverse stiffeners were performed in four girder tests and various non-destructive testing methods as well as post-weld treatment applied. Horizontally lying shear studs were tested in 7 dynamic and 3 static tests in cracked concrete. Different bridge coating systems were analysed in order to capture the influence at the steel-concrete interface. Different carbonation models were worked out. The results were integrated in the overall analyses.

Typical for the application range of steel and steel composite bridges three bridge types were identified according to span lengths and their functionality. A complete design following the rules given in the Eurocodes was performed and the integral approach of LCC, LCA and LCP applied. In order to allow for comparisons and elaboration of advantages a multitude of variants were studied including also typical concrete solutions in order to identify the chances and advantages for steel and steel-composite bridges if not only bid costs, but aspects of sustainability and life-cycle considerations are taken into account.

A workshop was organised in Paris, being a central location and easy to reach. Engineers from design offices and road authorities were invited and assisted the workshop for free. Among the research partners it was decided that after giving an introduction of the methodology and methods a feedback was requested from the participants in order to integrate the ideas in the ongoing of the research. The applications to the case studies were presented and discussed during the afternoon. Data were provided on a CD. The discussion especially with bridge authorities showed an increased interest in weathering steels and their positive effects on an improved durability and reduced maintenance costs.

The applied methodology is implemented in a user-friendly software tool which enables the calculation of case studies and the comparative analysis between alternative solutions.

In addition a handbook has been prepared, describing the life-cycle performance and maintenance strategies, the life-cycle assessment and the life-cycle costs analyses along the life-cycle of bridges in a concise way. The handbook should enable the direct application of the method by users by providing the necessary information in a condensed form. A case study and its variants are also given in the handbook in order to show an example and facilitate the use in practice.

Conclusions

This project aimed at providing a holistic approach by the combination of Life-cycle Assessment (LCA), Life-cycle Costs (LCC) and Life-cycle Performance (LCP) for the entire life-cycle of road bridges.

Hence, road bridges were looked at from different perspectives. Therefore a special feature of the project team was the very valuable involvement of universities, research centres, steel producers, design offices and especially road administrators and operators. This allowed gathering the knowledge and experience from partners originated in different European countries (different in regard of climate, bridge design policy and road administration, among others). During fruitful discussions this knowledge was brought together and evaluated. All the experience was gathered in a database to which many effort was put and which is a very important outcome of the project.

The investigations were not on theoretical bases but were carried out on realistic case studies of existing bridges. These case studies cover typical situations of road bridges.

The project team confirmed the importance of the inspection and maintenance scheme adopted during the operation stage of bridges. As this stage comprises 100 years and more, very decisive environmental and economic inputs are generated within this stage. It was agreed among the project partners on a standard European solution for inspection and maintenance actions. Two additional scenarios were elaborated and the importance of the inspection and maintenance strategies furthermore highlighted. The basis for efficient scheduling of inspection and maintenance actions is the knowledge about the degradation processes bridges are affected from. Here some valuable results could be achieved by detailed investigations and prototype testing on fatigue, corrosion and carbonation.

Bringing together the three aspects in a holistic approach, some general tendencies can be concluded:

- For the analysed environmental categories the impacts are dominated by the material production stage and the operation stage. During the operation stage most environmental impact is caused due to traffic congestion.
- Traffic interferences can lead to such high user costs that these can be by far the decisive aspect.
- The holistic approach opens up for long-term investments such as high strength steel (HSS), self-weathering steel or post-weld treatment, as these pay off easily during the operation stage.

Some more detailed conclusions for the single bridge types and case studies can be given in the following:

A- *Small motorway bridges*

For this bridge type it can be concluded that independently from the weighting scenario, providing a third lane in each traffic direction leads always to a better overall performance. Traffic congestion due to maintenance work causes major environmental impacts and user costs.

B- *Crossings of motorways*

The designs for crossings of motorways were compared for standard steel-composite bridges, concrete bridges and composite bridges with integral abutments and three-span bridges. As the variants of this case have different geometric characteristics a comparison was made per square meter of bridge area. Surprisingly the combination of the different criteria observed a best ranking for the three-span bridges. In a traditional point of view, based on the construction costs the concrete bridge cast in place would have been chosen. The ranking of the different bridge variants according to life-cycle costs and according to life-cycle environmental analysis is the same. Once the user costs are taken into account, the ranking is changed in the way that the integral steel-composite bridge is preferred, the usual steel-composite bridge last and the concrete bridge in the middle of both composite bridges. To conclude, it appears that for such short spans, integral abutments should always be preferred to usual abutments (with bearings and expansion joints). Also the choice between a concrete bridge and a steel-concrete composite bridge is governed by the importance given to user costs and therefore to the position of the bridge in the transport network.

C- *Big motorway bridges*

For big motorway bridges the two-deck solution comes to the best ranking compared to one large box-section. The diversion of traffic during maintenance actions onto one on of the two decks reduces user costs significantly. This shows that even the most expensive variant regarding the initial costs can be preferred with an integral approach.

It has been recognised that the idea of sustainable design also needs the transfer to practice. In order to allow for an easy access to the research results a handbook was elaborated. In addition a user-friendly software tool incorporates the databank and enables the calculation of LCA and LCC for bridges and the comparisons of alternative solutions by means of a multi-criteria decision analysis. For further spread and development also a succeeding dissemination project would be very beneficial in order to achieve a shift from the cost-effective to a life-cycle-effective mode.

Exploitation and impact of the research results

As a result of this research project first integral approach was applied to steel-composite bridges across Europe. Three main bridge types covering a majority of European bridges and a broad variety of variants were analysed in a holistic approach over the entire life-cycle. For future bridge design of new bridges, and also for inspection and maintenance scenarios an extensive databank was established. As the designed different case studies and data compilation were undertaken by project partners from different countries the input was collected all over Europe. During the project it was noticed by the project partners, that in general there is a rising public interest in sustainability over Europe. The main investigations are so far however carried out for buildings. Very different criteria apply for bridges, as bridges are long-living and for the infrastructure network very important. The actual needs and interests of bridge designers, planners, road authorities and ministries were gathered during the project runtime. This was especially observed and discussed during the SBRI-workshop organised and held in Paris (being easy to be reached) close to the end of the project, [41]. During the discussions participants from different backgrounds showed an active interest in the holistic sustainability approach and the applications to the case studies. The need was also underlined by the invited speaker from the French Ministry of Ecology, Sustainable Development and Energy (MEDDE). The effective use of self-weathering steel was discussed, [74]. The preliminary project results were given on a CD to the participants.

At the International Conference on Bridge Maintenance, Safety and Management Resilience and Sustainability IABMAS held in Stresa, Italy, a Special Session “Sustainability assessment of bridges” was organized by the SBRI-coordinator. With three presentations and papers the results were presented in the auditorium, showing the importance given to this session and topic, [15], [56], [55].

The Coordinator University of Stuttgart and the German Federal Highway Institute (BASt) are members of the German working group directed by the German Federal Ministry of Transport, Building and Urban Development (BMVBS) having the mandate to elaborate design supports for sustainable built road bridges. The SBRI project gave several inputs to this working group and the progress was presented continuatively at regular meetings as well as at a workshop organized at the ministry BMVBS directly, [24].

Brisa is the largest highway concessionary in Portugal, with interests in other road concessions in the world, so the output of the SBRI-project is of great importance for infrastructure management, with special emphasis on bridges and viaducts integrated in their highway networks. In a workshop on Eurocode 4-2 in Stockholm the research carried out in SBRI was presented to engineers and designers of mainly the northern European countries and a paper is now accessible in [21].

Project partners (FCTUC and USTUTT) were part of the European COST Action C25 “Sustainability of Construction”. A first direct involvement of the SBRI research was undertaken in the Training School for young researchers and the applied methodology presented in 2009. At the final C25 conference in 2011 first results were brought into the COST Action [23]. The current need for sustainable design was underlined.

As in this project the different national background of each partner and diverse climate situations for degradation processes on bridges were brought together and widespread knowledge and experiences in bridge maintenance have been collected, this will facilitate to reach acceptance for sustainable bridge design all over Europe.

During the project term more than 36 background documents were prepared, presenting the details of the work described in the final report. They are contained on the CD attached to the Final Report. During the project the basis has been set for doctoral, master and diploma thesis to be finalised in the near future, see e.g. [57]. The need for holistic and sustainable bridge design will also be included in the education of master students at the involved universities.

Besides the here mentioned activities further publications are in direct relation with this project and can be found in the own reference list under section 9.1.

As comparisons were made not only for variants of steel-composite bridges, but also for small span crossings with concrete bridges the competitiveness of steel-composite bridges if sustainability and an holistic life-cycle approach are considered could be demonstrated.

First ideas about a continuation and extension of the research work by a future European dissemination project have been discussed between partners. Based on the developed strategies, models and case studies analysed in this project and on the results achieved, further realised bridge structures could be investigated also to elaborate the differences and advantages of sustainable bridge design. By translations of the developed handbook [40] and SBRI-tool more engineers and authorities could be reached. Dissemination by workshops/seminars in different countries not only for selected experts but a wide spread of practising bridge engineers may finally lead to a wider acceptance of a necessary shift from the construction-cost dominated tender stage to a holistic life-cycle design for bridges across Europe.

1 Introduction

1.1 General

The current situation in the European bridge market is dominated by concrete bridges. Steel and steel-composite bridges only form an interesting alternative if additional criteria count such as e.g. aesthetics, construction time or reduced overall height. That is why the choice of orders is mostly only made according to minimum costs. However, with rising traffic volume and increasing vehicle gross weight this approach does no longer seem to be adequate, especially considering that bridges are in general long-living structures where the life-cycle is planned for more than 100 years.

Therefore, a new holistic approach was investigated by combining analyses of Life-cycle Assessment (LCA), Life-cycle Costs (LCC) and Life-cycle Performance (LCP). For steel-composite bridges innovative solutions were analysed to give alternatives to concrete bridges. Throughout the project the approach is applied to three realistic case studies and a multitude of variants representing standard situations of steel-composite bridges.

1.2 Sustainability by life-cycle design of bridges

Looking at bridges with a view to sustainability, not only the construction stage must be taken into account but the entire life-cycle of 100 years. These long-living structures are facing different degradation processes throughout the years. Degradation can be divided into several processes such as fatigue, corrosion and carbonation having an impact on various details. The structural function of the details, and therefore the structure itself, can be preserved and improved by maintenance and/or renewal actions concerning defects discovered during inspections, see Figure 1-1.

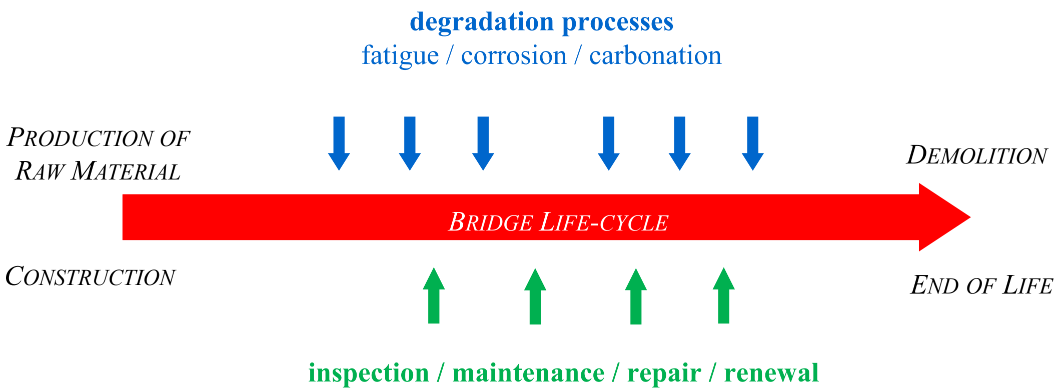


Figure 1-1. Schematic representation of the life-cycle of a bridge.

The life-cycle performance of steel-composite bridges is analyzed from the production of raw materials and the construction over the operation of the bridge (including maintenance etc.) till the demolition at the end-of-life. Under the participation of scientists, bridge owners, consultants and industry as project partners the life-cycle performance of steel-composite bridges was analyzed with a holistic approach as described in the following.

1.3 Holistic Approach

Life-cycle analysis aiming at sustainable bridge structures is divided into three main categories of consideration, see Figure 1-2. First, the environmental quality represents the analyses of emissions within the life-cycle assessment (LCA). Parallel, the economic quality comprises costs occurring during the entire life-cycle (LCC). The social and functional quality is involved as the third main category of life-cycle analyses. Applying this holistic approach to the entire life-cycle of bridges, influences affecting the structure at all stages are covered and no single design criterion is focused.

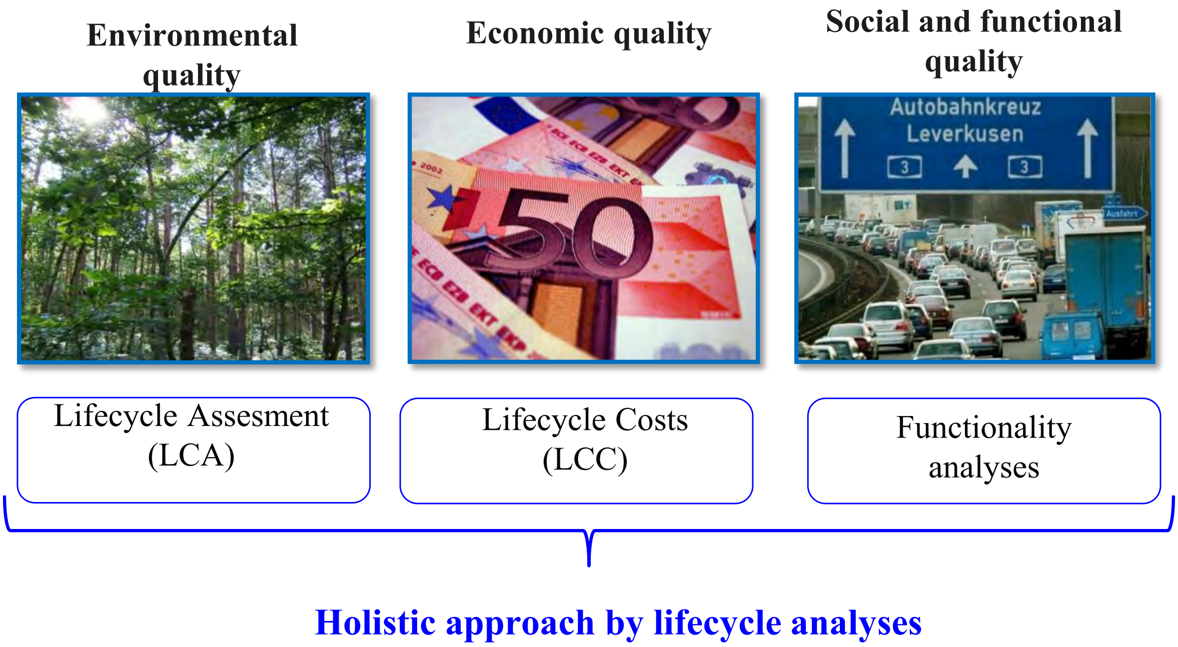


Figure 1-2. Holistic approach to life-cycle analyses.

The description of the life-cycle performance (LCP) of the structure and its details is the all-embracing condition to determine any inspection measurement during operation needed to guarantee a functional structure. The initial design and construction strongly interacts with the inspection and repair measurements needed during the service life and the scenario at the end-of-life of bridges. Possible effects of degradation and renewal actions may lead to additional emissions (LCA), costs (LCC) and restricted social and functional quality.

The application of this holistic approach over the entire life-cycle is the basis for a shift from bridge designs based on construction costs to a sustainable design taking into account the advantages of steel-composite bridges such as construction time, durability and exploration of material properties in an efficient way.

2 INFORMATION AND DATA FOR THE LIFE-CYCLE ANALYSIS OF BRIDGES

2.1 Data collection

2.1.1 *General*

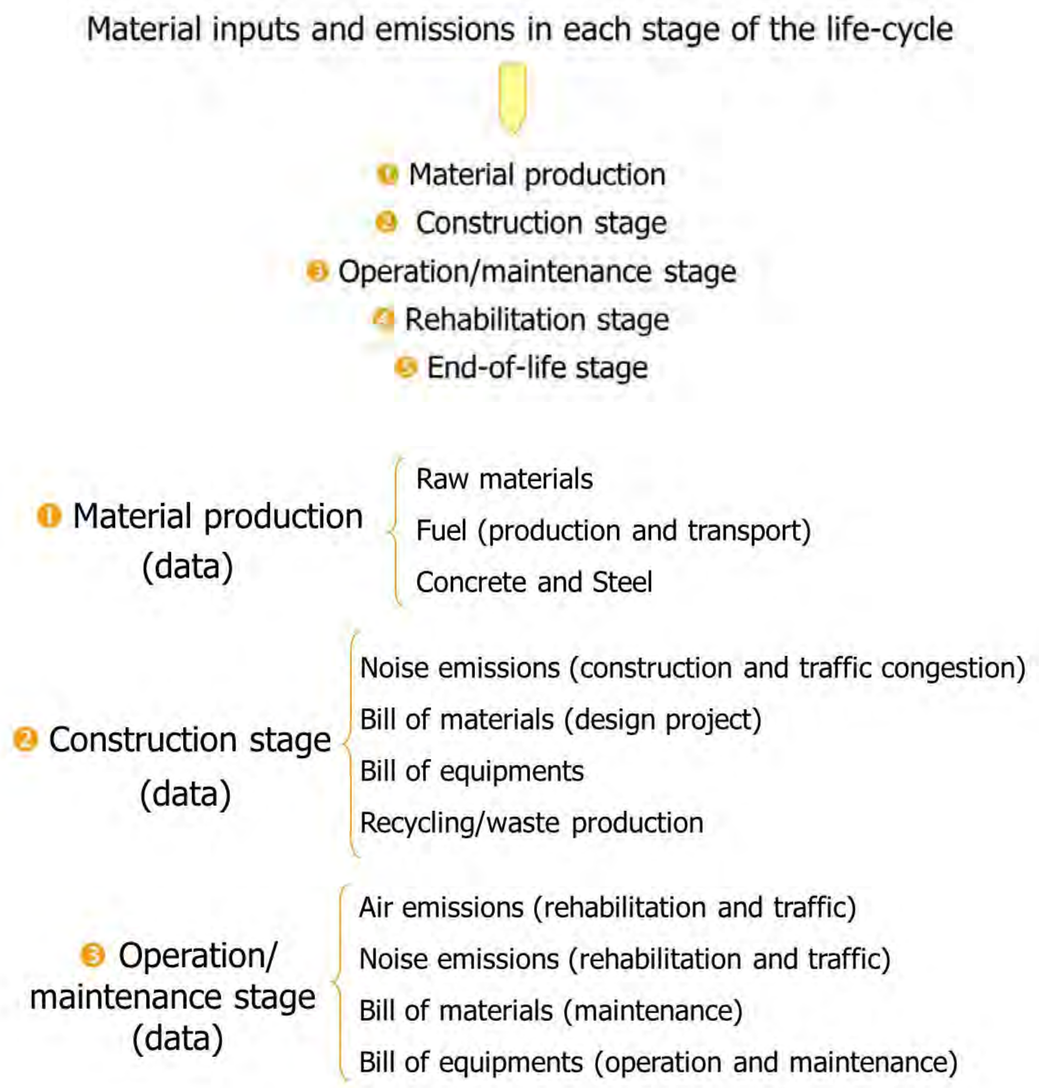
The basis of further tasks within this project was the gathering of information and data for the evaluation of life-cycle analysis, such as:

1. environmental analysis - LCA
2. costs analysis - LCC
3. performance analysis - LCP

In this research project, all the information was gathered in a database created specifically to collect the existing information and this database was also prepared to collect further data in order to meet also new needs in the future.

In order to systematize the information for each type of analysis, an evaluation of the type of information needed was made, and the following sections detail how the search for information was organized.

2.1.2 *Environmental analysis - LCA*



4 Reabilitation stage (data)

- Air emissions (rehabilitation and traffic)
- Noise emissions (rehabilitation and traffic)
- Bill of materials (design project)
- Bill of equipments
- Recycling/waste production

5 End-of-life stage (data)

- Air emissions (demolition)
- Noise emissions (demolition)
- Recycling/waste production

2.1.3 *Costs analysis - LCC*

Costs in each stage of the life-cycle

1. Construction stage
2. Operation/maintenance stage
3. Rehabilitation stage
4. End-of-life stage

1 Construction stage (data)

- Cost of materials
- Cost of construction processes

2 Operation/ maintenance stage (data)

- Operation and maintenance costs
- User and agency costs

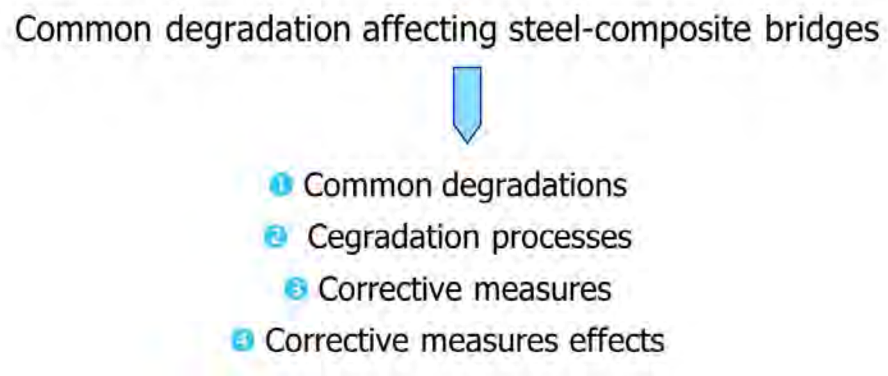
3 Rehabilitation stage

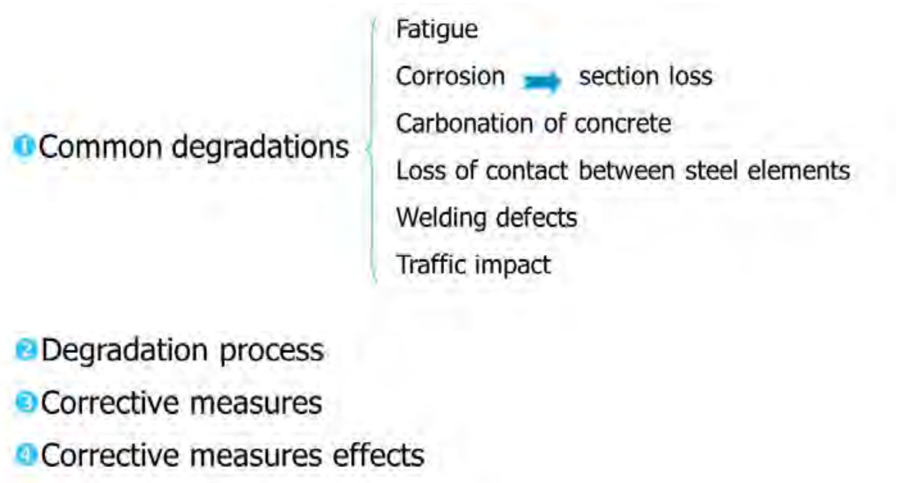
- Rehabilitation costs

4 End-of-life stage

- Demolition costs

2.1.4 *Performance analysis - LCP*





Some of the data to be used for the LCA analysis and LCC analyses are the same.
In order to systematize the search for information, in the database, specific sheets gather the information for each of the following life-cycle stages:

1. material production stage,
2. construction stage,
3. operation/maintenance stage,
4. rehabilitation stage,
5. end-of-life stage.

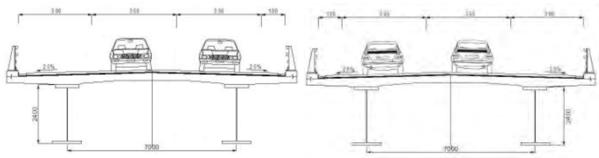
2.2 Case Studies

2.2.1 *Selection of bridge types*

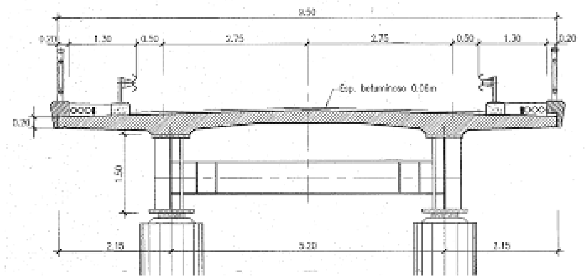
As steel-concrete composite bridges are currently built in a lot of various situations (from the bridge crossing a highway to the very long span highway bridge), and of various possible designs, it has been decided to work separately on three representative bridge types, for each of which a special design is classically preferred. Each type is defined by a span length or even a span distribution that the bridge families are designed for. They are also defined by the road supported by the bridge.

Small motorway bridges spanning around 50 to 60 meters using composite girders are considered as bridges of type A. Bridges of type B are crossings of motorways allowing comparisons between different span distributions (integral, two-span and three-span bridges). Span lengths up to 120 meters are reached by big motorway bridges and are assigned to type C with box-girder composite sections, compare Figure 2-1.

Type A bridges:



Type B bridges:



Type C bridges:

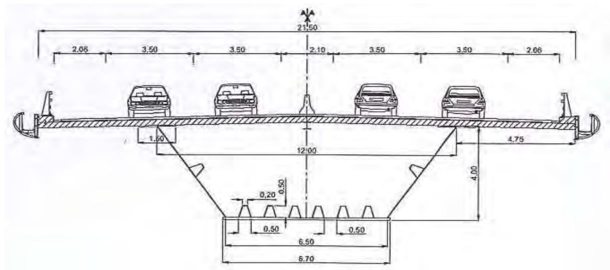


Figure 2-1. Steel-composite bridge of type A, B and C.

For precise investigations of the bridges for each bridge type *real* case studies were analysed and existing data collected. The following list (Table 2-1) summarizes the case studies and case studies variations, regarding only motorway bridges, selected for this research project. As real cases were analysed some minor adoptions for reasons of comparisons were undertaken.

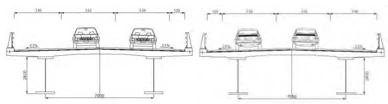
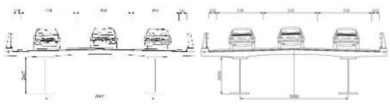
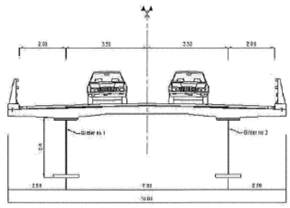
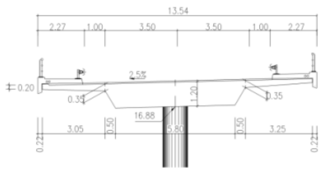
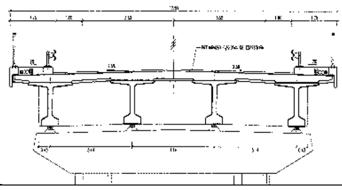
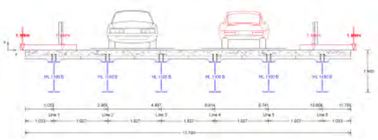
In case A, the variations include the increase of traffic by the introduction of an extra traffic lane, the application of improved fatigue resistance by post-weld treatment, the use of high strength steel (HSS) and self-weathering steel, specially to study the impact it can have on the materials (quantities) and impact on maintenance and repair actions.

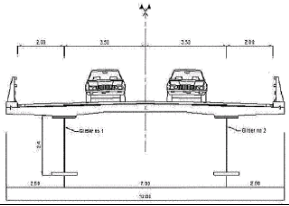
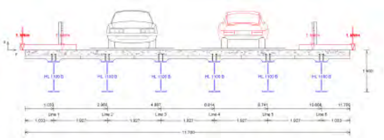
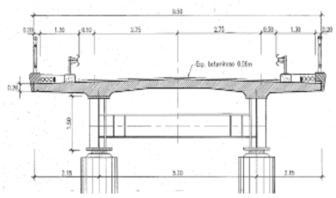
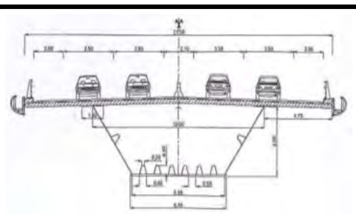
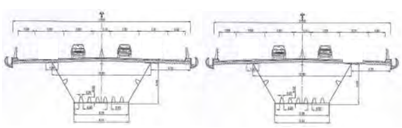
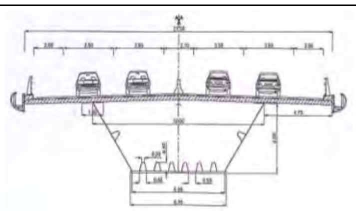
In case B, variations include considering integral abutments, considering one span and comparison to concrete bridges, namely with bridges with two cast in place girders and a bridge with four precast concrete girders.

Regarding case C, variations using high strength steel were studied also to analyse the impact it can have on the materials (quantities) and influence on maintenance and repair actions and therefore on the life-cycle analysis. The variant of two superstructures instead of one is considered as this allows for the possibility to close one superstructure completely during maintenance and/or rehabilitation and keep the other one for traffic.

Cases A0, B0.1 and C0 are the standard case studies for each type of motorway bridges. Table 2-1 shows the three types of bridges and the variations studied for each type.

Table 2-1. Project case studies.

	Case Studies	Cross-Section	nº spans span length (m) nº lanes width (m)	Comments
A0	**Small motorway bridge** continuous twin girder with prefabricated slab and girder, rolled or plated steel SETRA		3 spans 50-60-50 m 2 lanes in each direction width: 2 x 11 m	standard
A1				use of high strength steel S355/S460
A2				use of self-weathering steel
A3			3 spans 50-60-50 m 3 lanes in each direction width: 2 x 11 m	allowing the increase of traffic by providing three lanes
A4				improved fatigue design
B0.1	**Bridge crossing a motorway** continuous twin girder with prefabricated slab and girder, plated steel SETRA		2 spans 22.5 – 22.5 m 2 lanes width: 12 m	standard
B0.2	**Bridge crossing a motorway** reinforced cast in place concrete BRISA		2 spans 22.5 – 22.5 m 2 lanes width: 13.1 m	cast in place for comparison with case B0.1
B0.3	**Bridge crossing a motorway** reinforced precast concrete BRISA		2 spans 27 – 27 m 2 lanes width: 12.50 m	precast for comparison with case B0.1
B0.4	**Bridge crossing a motorway** continuous twin girder with prefabricated slab and concrete cross girders, rolled steel ARCELORMITTAL		2 spans 22.5 – 22.5 m 2 lanes width: 11.7 m	use of steel S355
B0.5	**Bridge crossing a motorway** continuous twin girder with prefabricated slab and girder, rolled steel ARCELORMITTAL		2 spans 22.5 – 22.5 m 2 lanes width: 11.7 m	use of high strength steel

	Case Studies (cont.)	Cross-Section	nº spans span length (m) nº lanes width (m)	Comments
B1.1	**Bridge crossing a motorway** prefabricated slab and girder, plated steel SETRA		1 span 40.8 m 2 lanes width: 12.50 m	considering use of integral abutments and elimination of internal support → 1 span bridge
B1.2	**Bridge crossing a motorway** prefabricated slab and girder, rolled steel ARCELORMITTAL		1 span 40.8 m 2 lanes width: 11.7 m	
B1.3	**Bridge crossing a motorway** prefabricated slab and girder, plated steel USTUTT		1 span 40.8 2 lanes width: 12,50 m	additionally considering horizontally lying shear studs
B2.1	**Bridge crossing a motorway** prefabricated slab and girder, rolled cross beams, plated steel BRISA		3 spans 19-41-19 m 2 lanes width: 10 m	use of two counter-weight spans
B2.2	**Bridge crossing a motorway** prefabricated slab and girder, rolled cross beams, plated steel SETRA		3 spans 19-41-19 m 2 lanes width: 12.5 m	Additionally use of self-weathering steel
C0	**Motorway bridge** box girder composite section SETRA			standard
C1.1	**Motorway bridge** box girder composite twin section SETRA		5 spans 90 - 3x120 – 90 m 2 lanes each direction width: 20 m	two separated sections
C1.2	**Motorway bridge** box girder composite section SETRA			use of high strength steel S460/S690

2.2.2 *Gathering of information for the case studies*

For each case study, information was gathered from different project partners and was used for comparison and for decision of which data should be used and be adequate for this research project, as for example:

- bills of materials from bridge projects in different countries provided by different project partners;
- units and unit conversion for coherent information, used to measure some structural and non structural elements of the bridges (that are the most commonly used and are mentioned in the literature database - [80], [121], [128]);
- materials used (steel and concrete) were analysed under the construction regulations prevailing in project partners countries.
- key structural and non-structural items object of this study;
- non-structural elements/equipments to consider in the study.

In a second phase, costs associated with the bill of materials were gathered and studied, and the conclusion was that in some cases the prices per unit could have a big variation. To study in more detail the cause of this discrepancy, a list with capacity rates for the works that interest this project was made to assess these differences, considering for example that capacity rates for new construction and minor repair works could be different.

Regarding the costs (regardless of the life-cycle stage) the differences between the hourly payment rates, materials, material production, etc, are also different from country to country, and a list of detailed issues like unit costs for materials, unit costs for capacity rates and costs for skilled labour was prepared, for comparison and analysis.

Detailed data related to the production and construction stages was collected for two specific case studies with the aim of defining which principal elements should be considered in this project for the study of different life-cycles (LCA, LCP and LCC).

Regarding the standard case studies A0, B0 and C0, it was also necessary to define the following issues to study the bridge life-cycle analysis, so the database includes the following data:

- Air and noise emissions (equipment) – to study environmental impact during construction, maintenance actions and in the end-of-life stage (demolition);
- Equipment emissions – for the environmental impact due to production, construction, rehabilitation and end-of-life stage;
- Traffic distribution and traffic evolution - on the different types of bridges to study environmental and social impacts;
- Definition of the main structural / non structural bridge elements to consider – because of differences between the case studies;
- Definition of the average service life for each structural / non structural elements (based on real data maintenance actions in different European countries) – important for the definition of maintenance / repair actions Table 2-3;
- Material quantities – necessary for the construction of each case study bridge– important to study environmental impact, costs related to construction, inspection and maintenance actions among others;
- Types of inspection and inspection frequencies and costs - necessary for the definition of an inspection strategy during the bridge service life;
- Definition of standard maintenance actions for the structural / non structural elements, unit costs for each action and their capacity rates;
- Definition of maintenance actions average service life – necessary for the definition of a standard maintenance strategy;
- Traffic restrictions for the different types of inspection and during maintenance actions or repairs – important to study user costs inconvenience during the bridge service life.

It is important to say that for all of the information gathered an important work evaluating types of works, unit costs and capacity rates was done, since in different countries these items can have big variations and it was necessary to find an average and suitable value or range of values in order to accomplish the life-cycle analysis.

2.3 Inspection and maintenance strategies

2.3.1 *General*

As the time in service and operation comprises the highest amount of lifetime of a bridge an exact capture of this phase is most relevant. The operation phase contains two significant actions that are held with some frequency and have an important weight on the life-cycle costs analysis of bridges: Inspection and maintenance.

Bridge inspection activity is very important as it allows the study of the evolution of different damage and helps to define the adequate corrective measures to ensure structural safety throughout the bridge lifetime. Possible repair scenarios are to be adapted to annual budgets so that the required maintenance actions guarantee the quality condition of bridges. The aim of these maintenance actions is to ensure the performance and structural safety, for which the bridge was designed, taking into account the environment and the level of planned maintenance, considering a 100-year service life, as assumed in the criteria defined in EN 1990 [75], for bridges.

2.3.2 *Inspection*

Activities of bridge inspection in different countries in Europe can be of quite big diversity. The activities were analysed and compared, and a standard inspection strategy elaborated, compare the internal document about inspection strategies, [3]. Inspections (including the types and aims of inspection and the frequency for these inspections) were gathered and summarized, Table 2-2. This issue is important since it enabled the definition of the number, types and frequencies of inspections needed during the bridge service life of 100 years and lead to the compilation of a standard inspection strategy being applied to the bridges further analysed. In Europe, despite the different location, weather, type of bridges that require different frequencies for inspection actions, it is possible to define common types of inspection and frequencies. These scenarios may not be exactly fulfilled everywhere but are somehow representative at least for the countries of the project partners involved. In the standard inspection strategy (assumptions for this strategy are detailed in the internal document [3]) it is also assumed that the costs for each type of inspection includes the updating of all information registered on a "Bridge Management System" or other type of registry considered relevant, the emission / preparation of inspection reports, needs for traffic restrictions and the need for special inspection units.

Table 2-2. Inspection types and frequencies of the standard scenario.

Type of Inspection	**Frequency [years] case A, B,C**
Routine	annually
Main, principal or periodic special	6
Special or exceptional	twice during life-cycle

The definition and aim of each of the types of inspection are:

<u>Routine or annual</u> – This type of inspection consists of a visual observation and all information is collected on specific standard forms / check lists. The aim is to detect small damage that can be promptly repaired. The team is formed by one or two members of the maintenance staff with specific training.

Main, principal or periodic special – This is a detailed visual inspection, performed with specific inspection units for access. Disorders are marked on the bridge (crack openings, areas with damaged concrete, exposed corroded rebars). The aim is to confirm the initial /last/ latest condition rating of the bridge and help define the need of repair actions. The team will be lead by an engineer or experienced person in bridge inspection.

Special, extra or exceptional detailed – This is a more detailed inspection carried out when:

a) a repair plan is needed for the complete rehabilitation of the bridge;
b) a specific damage needs to be assessed (natural hazards, vehicle collision, etc.).

Non-destructive / destructive tests are used together with laboratory analysis. The results of the tests help evaluate the damage conditions and allow recommendations for damage repair. The aim is to assess the structural condition or define the cause for a specific damage and set rehabilitation strategy. The team is led by an engineer or experienced person in bridge inspection (for at least 5 years).

In the Annex – **Table A 1** summarizes data that were assumed for the definition of a standard Inspection scenario, types and frequency for each inspection, average costs per bridge and m² and also traffic restrictions on the bridge when each type of inspection takes place.

2.3.3 *Maintenance*

A direct consequence of inspection actions are maintenance / repair actions. Regarding maintenance during the operation stage, a list with maintenance strategies was compiled for different European countries. Maintenance activities can be divided into categories regarding the intensity of maintenance. Minor works, done on a regular basis, such as cleaning of bearings, expansion joints and drainage systems, and the frequency for these actions and repair/rehabilitation actions for the repair of major defects due to slow deterioration or replacements that are done to ensure adequate structural condition (replacement of bearings, corrosion protection, etc.) were collected and compared. Detailed background information is prepared in the internal document [4].

For the standard maintenance strategy, after the definition of the structural / non structural elements of the bridges to be considered, the average service life for each of them was assumed based on the experience of project partners, compare Table 2-3. The definition of the frequencies of maintenance actions, that are associated with capacity rates and unit costs, allows the elaboration of an adequate maintenance strategy, Table 2-4. This strategy regards measures/actions that are recommended in order to maintain a good quality condition of the bridges. In this research project the assumptions and recommendations follow the actual Eurocodes.

Table 2-3. Average service life for bridge elements.

Structural/non structural element	unit	Average service life (years)
		Case A,B,C
Superstructure concrete	m³	100
Concrete edge beam	m	40
Safety barrier	m	40
Superstructure steel	t	100
Steel corrosion protection	m²	35
Expansion joints	m	40
Road surface	m²	20
Waterproofing layer	m²	40
Metal cornice gutter	t	25
Elastomeric Bearing	piece	35
Calote Bearing	piece	100
Railing	m	40

Table 2-4. Frequencies of maintenance actions for the standard maintenance scenario.

Structural / non structural element	**Maintenance actions**	**Frequency of maintenance actions [years]**
		Case A,B,C
Superstructure concrete	small area repairs	25
Concrete edge beam	minor repairs	25
Safety barrier	partial replacement	25
Steel corrosion protection	repainting of corrosion protection	25
Expansion joints	partial replacement	10
Road surface	minor repairs	10
Waterproofing layer	-	0*
Metal cornice gutter	-	0*
Elastomeric Bearing	cleaning, painting, lubricating	20
Calote Bearing	-	0*
Railing	painting	20

(*) – Elements with no maintenance actions. Total replacement takes place when the service life is reached

Information gathered also took into account the experience that project partners have on bridge assessment since it is one of their major activities, and also their registries on maintenance and rehabilitation activities on composite bridges and concrete bridges, were considered for comparison.

In the Annex – **Table A 2** details data that were assumed for the definition of a standard maintenance scenario (maintenance actions, maintenance actions frequency, average service life, capacity rates and distribution over the 100-year service life).
Table A 3 shows for each maintenance action the types of materials used, the units for these materials and the associated traffic restrictions.

2.3.4 *Additional scenarios*

Regarding inspection and maintenance activities, beside the standard scenarios, two specific scenarios were studied:

- *Lack of money* – in this scenario, it is assumed that along the bridge life-cycle, there will be not enough money to undergo the necessary maintenance/ repair actions and the bridge will be critically deteriorated on year 100, with traffic / weight restrictions. Inspection activity is increased and frequency of main, periodic or special inspection is reduced to five years, increasing the total number of inspections, especially in the last years of the bridge life-cycle with the introduction of more special inspections in order to allow the knowledge of the existing bridge condition, compare Table 2-5. The special inspections will take place in years 35, 50, 75 and 90. New maintenance actions are introduced to extend service life of some bridge elements in order to delay or remove some maintenance actions.

Table 2-5. Lack of money scenario – average maintenance/repair works frequency.

Structural/non structural element	Maintenance actions	Frequency of maintenance actions [years]
		Case A,B,C
Superstructure concrete	small area repairs	50
Concrete edge beam	minor repairs	50
Safety barrier	partial replacement	20
Steel corrosion protection	repainting of corrosion protection	25
Expansion joints	partial replacement	10
Road surface	minor repairs	10
Waterproofing layer	-	0*
Metal cornice gutter	-	0*
Elastomeric Bearing	clean, painting, lubricating	20
Calote Bearing	-	0*
Railing	painting	20

(*) – Elements with no maintenance actions. Total replacement takes place when the service life is reached.

In the Annex - **Table A 4** details the assumptions considered for the lack of money inspection scenario and **Table A 5** the data for lack of money maintenance scenario.

- *Prolonged life* – in this scenario, it is assumed that the decision of maintaining the bridge for an additional 30 years (130 years total of service life) is taken at year 80. So, inspection and maintenance strategies until year 80 will be the same as those assumed for the standard inspection/maintenance strategies. After year 80 inspection and maintenance actions are adapted in order to accomplish this service life extension and maintenance repair for some elements will be more frequent between years 115 and 130. In this scenario, it is also assumed that the steel superstructure will have no problems regarding fatigue, so no reinforcement actions regarding this problem were included in this scenario, see Table 2-6.

Table 2-6. Prolonged life scenario – average maintenance/repair works frequency.

Structural/non structural element	Maintenance actions	Frequency of maintenance actions [years]
		Case A,B,C
Superstructure concrete	small area repairs	25
Concrete edge beam	minor repairs	40
Safety barrier	partial replacement	20
Steel corrosion protection	repainting of corrosion protection	25
Expansion joints	partial replacement	10
Road surface	minor repairs	10
Waterproofing layer	-	0*
Metal cornice gutter	-	0*
Elastomeric Bearing	clean, painting, lubricating	25
Calote Bearing	-	0*
Railing	painting	20

(*) – Elements with no maintenance actions. Total replacement takes place when the service life is reached.

In the Annex - **Table A 6** details the assumptions considered for the prolonged life inspection scenario and **Table A 7** the data for prolonged life maintenance scenario.

2.4 Further considerations

All data referred in the previous sections are exclusive for steel composite bridges, but two case studies – B0.2 and B0.3 (see Table 2-1) concern concrete bridges. The aim of the inclusion of these case studies was to enable a comparison of the life-cycles to another universe of bridges that are common on motorways.

During the project and with all the information gathered all case studies were considered to enable the convenient comparison and help draw the conclusions for sustainable steel-composite bridges. So, the database includes different types of data, material quantities, costs for the different case studies. Thereby it is the best way to assess which is the most convenient design for a certain and specific situation. Also the comparison between countries was taken into account in order to study the principal differences regarding material and construction costs, and also different environmental conditions that can determine the need for different maintenance and repair strategies, including the periodicity of these actions.

More detailed information is given in the tables in Annex A and the internal background documents [4], [3]. This database has been essential for the realization of the various case studies and is implemented in the design tool.

3 CASE STUDY DESIGN

3.1 Bridge types

Bridges of type A are supposed to be representative for medium span bridges (from 40 m to 90 m span length) for which a twin girder design is classically preferred. Each bridge of type A supports a highway with two 3.5 m wide lanes per traffic direction. In each direction, the two lanes are bordered by a 3 m wide emergency lane on the right hand side and a 1 m wide left shoulder on the left hand side. The whole roadway is bordered by normalised safety barriers. The bridge has a symmetrical structure with three spans of 50 m, 60 m and 50 m (i.e. a total length between abutments of 160 m), see Figure 3-1.

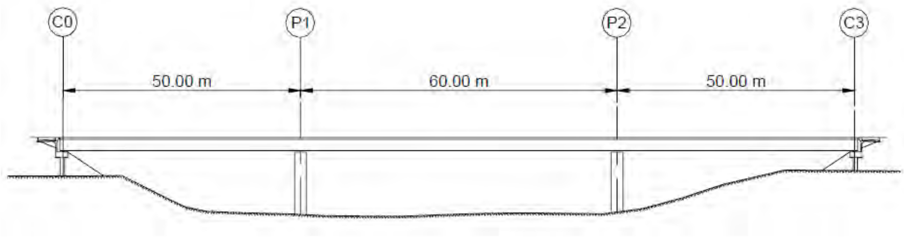


Figure 3-1. Case A: span distribution.

Bridges of type B are supposed to be representative for short span bridges (under 50 m) spanning over a motorway. The aim of this bridge type is to allow using improvement methods which can only be used for short span such as integral abutments. This type is defined by its total length but there can be variations of the span distribution between the studied examples. Each bridge of type B crosses a highway. It supports a smaller road made of two 3 m wide lanes: one per direction. Each lane is bordered by a 1 m wide right shoulder, a safety barrier, a footway (1.5 m wide) and a hand railing. The bridge crosses a 40 m wide highway. The span distribution can possibly take into account an intermediate support in the middle of the highway between the two directions of traffic. For this bridge type also typical solutions in concrete have been considered because short spans are a typical domain of concrete bridges where usually steel-composite bridges suffer from a harsh competition. However, by taking also the user costs of the crossed motorway into account the advantage of a quick and easy erection of steel-composite bridges and the reduced impact on the running motorway traffic for any maintenance work may bring advantage to the steel-composite bridge solutions.

Bridges of type C are supposed to be representative for long span bridges (from 70 m to 160 m span length) for which a box girder design is classically preferred. Each bridge of type C supports a highway with two 3.5 m wide lanes in each direction. In each direction, the two lanes are bordered by a 2 m wide emergency lane on the right hand side and a 1 m wide left shoulder on the left hand side. The whole roadway is bordered by normalised safety barriers. The bridge has a symmetrical structure with five spans of 90 m, 120 m, 120 m, 120 m and 90 m (i.e. a total length between abutments of 540 m), see Figure 3-2.

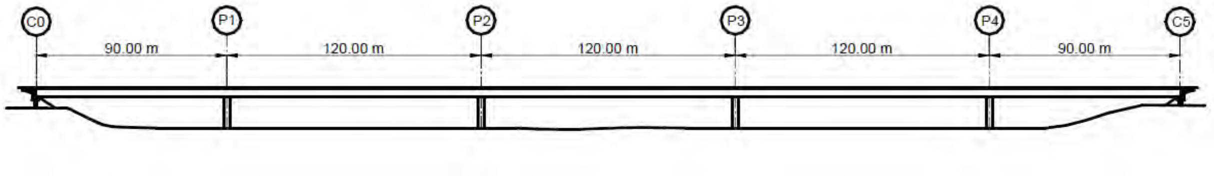


Figure 3-2. Case C: span distribution.

3.2 Type A (medium span bridges)

Reference case A0 has one superstructure per direction. Each superstructure is a steel-concrete composite twin-girder bridge, see Figure 3-3 [43]. The main girders are of constant height: 2400 mm. The lower flanges are 1000 mm wide whereas the upper flanges are 800 mm wide. The total slab width is 12 m. The centre-to-centre spacing between main girders is 7 m and the slab cantilever either side is 2.5 m.

For the construction, the structural steel is first installed by launching and then the 16 concrete slab segments (10 m long each) are poured on-site.

The standard design of case A0 leads to the elaboration of a list of outputs given in Table 3-1. The quantities refer only to the superstructure, and not the abutment, foundation, excavations or embankments. The quantity estimations do not consider the equipments necessary for construction (formwork, scaffolding, bracing, launching equipment), nor supplies related to the construction of the structure. Moreover, the mentioned quantities are the ones actually in place during the service life, and therefore do not include the scrap and waste associated with their development and construction.

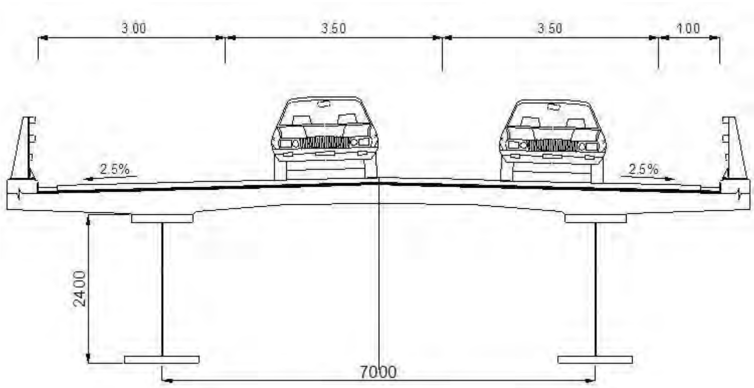


Figure 3-3. Case A0: cross-section.

Table 3-1. List of design outputs for A0 provided to LCA and LCC analysis.

	Quantity	*Unit*
Steel		
Structural steel (main girders + bracing frames) S355 N/NL	405 000	kg
Reinforcement steel bars f_{sk} = 500 Mpa – Concrete slab	124 000	kg
Reinforcement steel bars f_{sk} = 500 Mpa – Concrete support for the safety barrier	6 400	kg
Studs S235 f_u = 450	1 500	kg
Concrete		
Main slab C35/45	624	m^3
Concrete support for the safety barriers C35/45	32	m^3
Corrosion protection		
Paint class C4 ANV	3 000	m^2
Non-structural equipments		
Steel S235 JR (galvanised) – Safety barrier	20 800	kg
Waterproofing layer (3 cm thick)	1 792	m^2
Asphalt layer (8 cm thick - 1 760 m^2)	352	t
Metal cornice gutter - Aluminium alloy	8 000	kg
Comb expansion joint (range of opening: 85 mm)	24	m

Variant A1 consists of the substitution of the standard steel girders with high strength steel (grade S460) for the bridge main girders, [45]. High strength steel is located in the main girders flanges around intermediate supports.

The reduction of rigidity of the main girders due to the reduction of flanges thickness around intermediate supports requires stiffer transverse bracing frames. The addition of steel in transverse bracing frames curbed the global reduction of steel: this variant requires 51 000 kg of high strength steel, but finally allows to save 13.3 % of steel compared to Case A0.

Variant A2 consists of the substitution of self-weathering steel for structural steel, [48]. This variant allows cutting the need for painting and its maintenance. The extra thickness of steel for corrosion leads to increased quantities of steel: this variant requires 8.9 % more steel, but saves the whole corrosion protection.

Variant A3 consists of the consideration that due to traffic gowth, the bridge could be used with three lanes instead of two (this proposal comes from a risk analysis to anticipate the needs of the infrastructure manager in the medium or long term), compare the cross-section given in Figure 3-4, [46]. This variant allows reducing potential maintenance and strengthening actions or reconstruction of the structure (durability loss). Indirect costs generated in these cases may then be considerable. This variant requires 3.9 % more steel compared to A0.

Variant A4 consists of the same considerations as for variant A3 additionally the use of a post-weld treatment in form of Pneumatic Impact Treatment [91] on welds where fatigue governs the design, [49]. Only the welds of the vertical stiffeners on the lower flanges, around mid-span are treated. This variant allows reducing potential maintenance actions without increasing material quantities too much: this variant requires only 0.6 % more steel compared to A0 instead of 3.9% for variant A3.

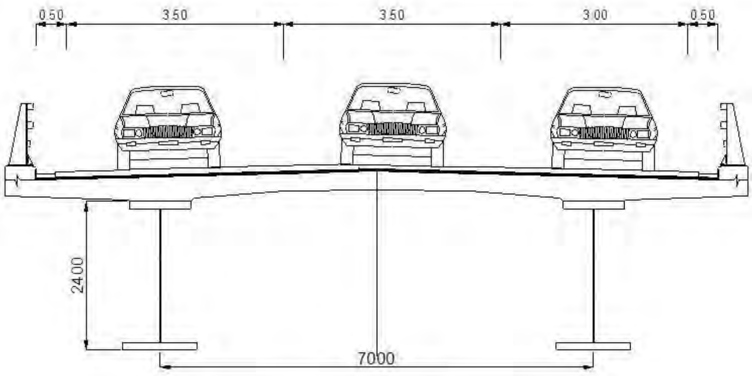


Figure 3-4. Cases A3 and A4: cross-section with unfavourable lane distribution.

3.3 Type B (short span bridges – crossings of motorways)

Type B is made up of 5 reference bridges (called B0.1, B0.2, B0.3, B0.4 and B0.5) and 3 variants (called B1.1, B1.2, B2.1 and B2.2). As for type A and case A0, Type B has a standard composite steel concrete bridge – B0.1 – to which variants are be compared. Nevertheless, Type B also has two other standard concrete bridges to which it seems interesting to compare the variants because concrete bridges are often preferred for such short spans. Bridges of Type B are all motorway crossings.

Reference case B0.1 is a steel-concrete composite twin-girder bridge, [50]. The bridge has a symmetrical structure with two spans of 22.5 m (i.e. a total length between abutments of 45 m) (Figure 3-5). The total slab width is 11.70 m. The centre-to-centre spacing between main girders is 6.5 m and the slab cantilever either side is 2.6 m, see the cross-section given in Figure 3-6.

For the construction, the structural steel is first installed with a crane and then the 23 pre-cast concrete slab segments (1.95 m long each) are installed and keyed.

The standard design of case B0.1 leads to the elaboration of a list of outputs given in Table 3-2. The quantities only refer to the superstructure, abutment, foundation, and not the excavations or embankments. The quantity estimations do not consider the equipments necessary for construction (formwork, scaffolding, bracing, launching equipment), nor supplies related to the construction of the structure. Moreover, the mentioned quantities are the ones actually in place during the service life, and therefore do not include the scrap and waste associated with their development and construction.

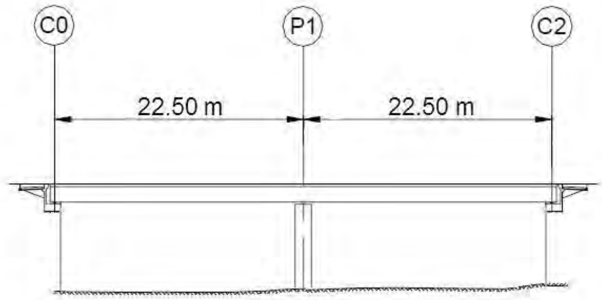


Figure 3-5. Case B0.1: Span distribution.

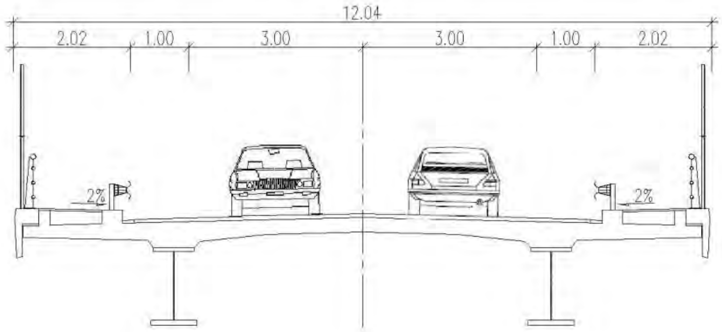


Figure 3-6. Case B0.1:Cross-section.

Table 3-2. List of design outputs for B0.1 provided to LCA and LCC analysis.

	Quantity	*Unit*
Steel		
Structural steel (main girders + bracing frames) S355 N/NL	63 500	kg
Reinforcement steel bars f_{sk} = 500 Mpa – Concrete slab	31 000	kg
Reinforcement steel bars f_{sk} = 500 Mpa – Concrete support for the safety barrier	5 700	kg
Studs S235 f_u = 450	680	kg
Concrete		
Main slab C35/45	152	m^3
Concrete support for the safety barriers C35/45	29	m^3
Corrosion protection		
Paint class C4 ANV	450	m^2

Non-structural equipments		
Steel S235 JR (galvanised) – Safety barrier	4 500	kg
Waterproofing layer (3 cm thick)	503	m²
Asphalt layer (8 cm thick - 360 m²)	72	t
Concrete cornice gutter – C25/30	12	m^3
Comb expansion joint (range of opening: 85 mm)	23.4	m
Supports		
Concrete C35/45	490	m^3
Reinforcement steel bars f_{sk} = 500 MPa	62 650	kg

Reference case B0.2 is a concrete bridge cast in place. The bridge has a symmetrical structure with two spans of 22.5 m (i.e. a total length between abutments of 45 m). The total slab width is 13.10 m.

Reference case B0.3 is a pre-cast concrete bridge. The bridge has a symmetrical structure with two spans of 27 m (i.e. a total length between abutments of 54 m). The total slab width is 12.50 m.

Reference case B0.4 is a steel-concrete composite multiple-girder bridge. Girders are made of steel grade S355. Girders are rolled girders HE 900 A. The bridge has a symmetrical structure with two spans of 22.5 m (i.e. a total length between abutments of 45 m). The total slab width is 13.40 m. The centre-to-centre spacing between main girders is 2.68 m and the slab cantilever either side is 1.34 m.

Reference case B0.5 is a steel-concrete composite multiple-girder bridge. Girders are made of steel grade S460. Girders are rolled girders HE 800 A. The bridge has a symmetrical structure with two spans of 22.5 m (i.e. a total length between abutments of 45 m). The total slab width is 13.40 m. The centre-to-centre spacing between main girders is 2.68 m and the slab cantilever either side is 1.34 m.

Variant B1.1 consists in the design of integral abutments with a 40.8 m single span, that is to say no support in the middle of the highway, compare the span distribution given in Figure 3-7, [51]. Main girders are made of plated steel.

This variant is 9.3 % shorter than case B0.1, but allows saving 11 % of structural steel and 21.5 % of concrete (mainly due to the elimination of the intermediate pier). Moreover, it eliminates some maintenance actions: replacement of expansion joints and bearings.

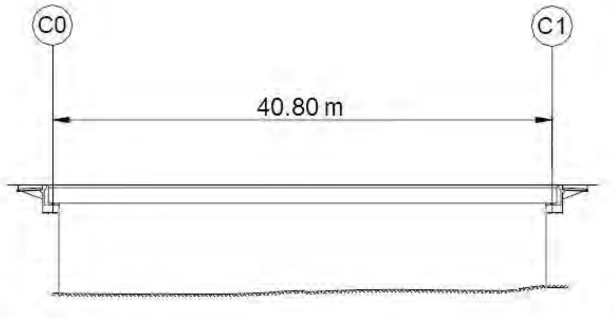


Figure 3-7. Case B1: span distribution.

Variant B1.2 consists of the use of integral abutments with a 40.8 m single span, that is to say the same length as B1.1. Main girders are made of high strength (S460) rolled steel. This variant is 9.3 % shorter than case B0.1, but requires 55.9 % more structural steel. Nevertheless, it eliminates the intermediate pier and some maintenance actions: replacement of expansion joints and bearings.

Variant B1.3 consists of an additional variant with integral abutments for the 40.8 m single span as regarded in B1.1 and B1.2. The variant is analysed with the application of horizontally lying shear studs [57] and therefore allows avoiding the upper steel flange including the welding detail between flange and steel. It also gives a very direct connection between steel girder and concrete slab creating prefabricated composite girders.

Variant B2.1 consists of the use of "counterweight" spans instead of integral abutments. The bridge has a symmetrical structure with three spans of 18.50 m, 40.80 m and 18.50 m (i.e. a total length between abutments of 77.80 m), see Figure 3-8. This variant allows building a central span with almost the same dimensions than the B1.1 single span, that is to say no support in the middle of the highway is needed. Integral abutments are replaced by simple abutments but the bridge is twice as long.

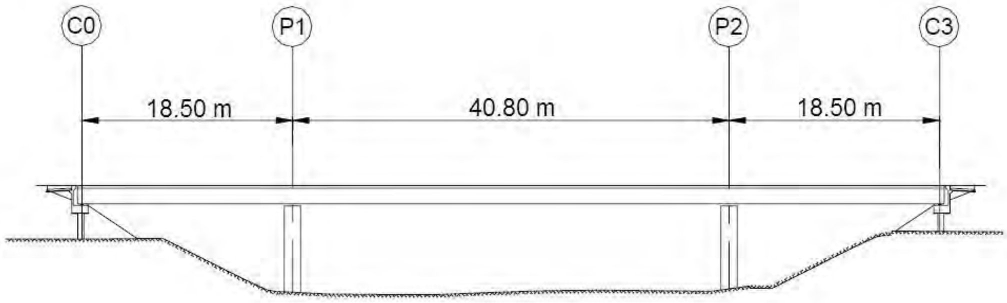


Figure 3-8. Case B2: span distribution.

Variant B2.2 consists of the substitution in variant B2.1 of self-weathering steel for structural steel. This variant allows cutting the need for painting and its maintenance. The extra thickness of metal for corrosion leads to increased steel quantities.

3.4 Type C (long span bridges)

Reference Case C0 is a single steel concrete composite box girder bridge. The main girder is of constant height: 4000 mm. The lower flange is 6700 mm wide whereas the upper flanges are 1800 mm wide. The total slab width is 21.50 m. The centre-to-centre spacing between main girders is 12 m and the slab cantilever either side is 4.75 m, Figure 3-9.

For the construction, the structural steel is first installed by launching and then the 45 concrete slab segments (12 m long each) are poured on-site.

The standard design of case C0 leads to the elaboration of a list of outputs given in Table 3-3. The quantities refer only to the superstructure, and not to the abutment, foundation, excavations or embankments. The quantity estimations do not consider the equipments necessary for construction (formwork, scaffolding, bracing, launching equipment), nor supplies related to construction of structure. Moreover, the mentioned quantities are the ones actually in place during the service life, and therefore do not include the scrap and waste associated with their development and construction.

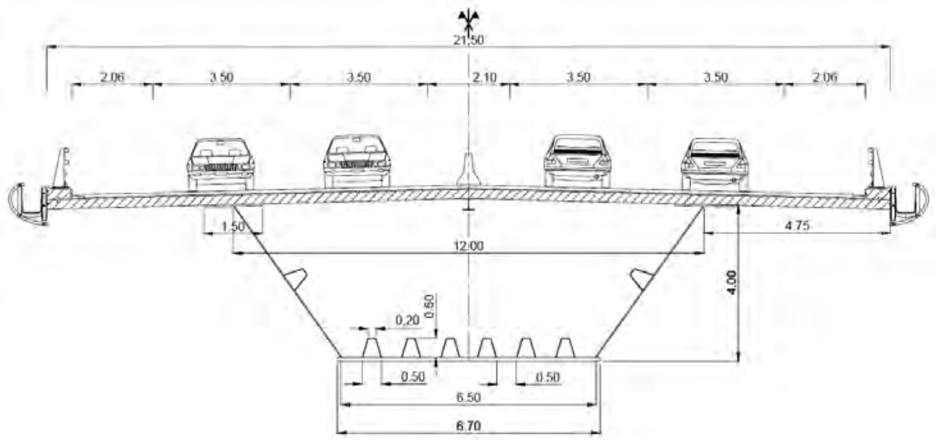


Figure 3-9. Case C0: cross-section.

Table 3-3. List of design outputs for C0 provided to LCA and LCC analysis.

	Quantity	*Unit*
Steel		
Structural steel (main girders + bracing frames) S355 N/NL	5 469 000	kg
Reinforcement steel bars f_{sk} = 500 Mpa – Concrete slab	781 000	kg
Reinforcement steel bars f_{sk} = 500 Mpa – Concrete support for the safety barrier	25 000	kg
Studs S235 f_u = 450	16 000	kg
Concrete		
Main slab C35/45	2 903	m^3
Concrete support for the safety barriers C35/45	173	m^3
Concrete for the central safety barriers C35/45	131	m^3
Corrosion protection		
Paint class C4 ANV	10 095	m^2
Paint class C3 ANI	27 198	m^2
Non-structural equipments		
Steel S235 JR (galvanised) – Safety barrier	70 000	kg
Waterproofing layer (3 cm thick)	11 189	m^2
Asphalt layer (8 cm thick)	2 096	t
Metal cornice gutter - Aluminium alloy	22 000	kg
Comb expansion joint (range of opening: 85 mm)	43	m

Variant C1.1 consists of the construction of one superstructure per traffic direction. Because of the span length, each superstructure is made of a steel-concrete composite box girder. The construction of two separate superstructures creates additional effort at the construction stage. This variant requires notably 14 % more structural steel and 16.4 % more concrete, but allows the closing of only one superstructure during any maintenance actions.

Variant C1.2 consists of the substitution of hybrid girders with high strength steel (grades S460/S690) for the bridge box-section. Steel S690 is located in the box girder flanges around intermediate supports. This variant requires 181,000 kg of steel grade S690 and 3,081,000 kg of steel grade S460, but allows saving 23 % of steel compared to Case C0.

3.5 Further considerations

The three selected bridge types A, B and C cover a majority of European standard road bridge situations. By the analysis of variants comparisons and conclusions are drawn and further described in the holistic approach. The variations include the use of high strength steel, self-weathering steel and improved fatigue design for type A bridges. It also considers the option of an additional 3rd lane. In addition to the variance of the material strength the span distribution is changed from two span (B.0) to single span (B1) and to three span bridges for type B. The big motorway bridges of type C allow besides the change of material strength for comparisons between a single deck or a double deck cross-section. The realistic design of standard cases and variants defines material masses and dimensions which form the basis of the further LCA and LCC analysis.

The detailed designs of the bridges are given in the background documents.

4 LIFE-CYCLE ASSESSMENT AND LIFE-CYCLE COSTS

4.1 General procedures

An integral life-cycle approach for the assessment of motorway bridges was developed in the framework of this project. The aim of the approach is the life-cycle assessment of a bridge in the context of sustainable development and, in particular, in the context of sustainable construction. Therefore, the approach aims at balancing environmental and economic aspects.

Currently, there is not a standardized methodology providing guidance for an integral life-cycle analysis of a construction system [93]. The life-cycle environmental analysis has currently the most well established standardized framework, although there is still no generalized acceptable methodology in the scientific community. In a decreasing order of development follows the life-cycle economic analysis. For this reason, the development of the general framework for the integral life-cycle analysis was based on the standardized framework for Life Cycle Environmental Analysis (LCA), according to the series of ISO standards 14040 [103], with further adaptation in order to include economic criteria.

Therefore, the generalized framework proposed in this chapter entails the four main steps of the ISO standard 14040 [103]: the goal and scope step; the inventory step; the impact assessment step; and the interpretation step. However, as already referred, each step of the analysis was adapted in order to allow the integration of economic aspects in the life-cycle analysis.

The traditional design of a bridge is based on the requirements of codes and rules that have been developed for that purpose. These requirements are related to the safety of the structure and usually include rules for resistance, durability and serviceability. In this approach, the initial safety of the structure, according to the requirements of the Structural Eurocodes, is assumed to be fulfilled. In addition, it is assumed that major failure of the bridge does not occur over the time span of the analysis (100 years). However, in a life-cycle analysis, this is not enough. Bridges start to deteriorate immediately after they enter into service. The rate of deterioration depends on many factors for different types of bridges. In order to keep the bridge above the required condition, maintenance and rehabilitation of the bridge is required. Each time an intervention in needed on the bridge, it originates environmental and economic impacts that need to be considered in a life-cycle analysis. Hence, the life-cycle environmental and economic analyses of bridges are directly dependent on the lifetime structural performance and this relationship is addressed in the methodology, as illustrated in Figure 4-1 by integrating the structural performance of the bridge over the lifespan of the analysis.

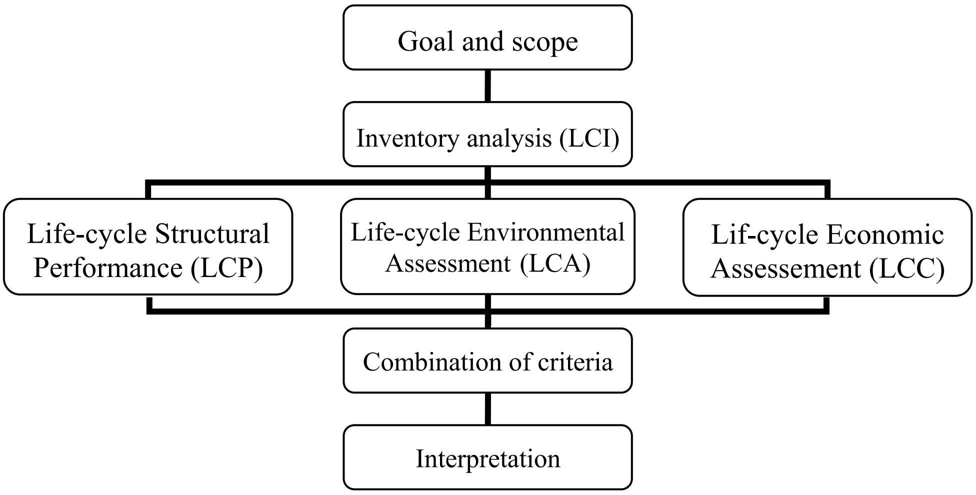


Figure 4-1. Flowchart of the Life Cycle Integral Analysis.

The integration of the three criteria is based on the condition that the evaluation of the criteria shares the same goal and scope and that they are based on the same inventory analysis.

The impact assessment stage of the three main categories is made separately for each criterion although based on the same assumptions. The following step, the combination of criteria, depends on the aim of the analysis. On the one hand, if the aim of the analysis is to identify the improvement possibilities of the processes contributing with major impacts, then the structural, environmental and costs performances may be interpreted individually. On the other hand, if the aim of the analysis is to help in a decision making problem, then the balance between the individual performances may be achieved by a multi-criteria decision analysis. It should be emphasized that a life-cycle analysis is not a decision making approach; however, it can provide valuable information for decision makers in the process of decision making [93].

4.2 Life-cycle Assessment

4.2.1 *Introduction*

Sustainability requires life-cycle thinking. In the context of sustainable construction, the design of a bridge goes beyond the traditional requirements of safety and initial costs. It comprehends the life-cycle of the bridge, from raw material acquisition to the bridge's decommissioning [93]. This implies the prediction of the structural behaviour of the bridge over its life span, the estimation of bridge maintenance and repair, etc. Moreover, non-traditional aspects of environment, economy and society shall be considered together with traditional ones and currently, most engineers are not prepared for these new requirements.

Life-cycle analyses are usually time consuming and thus costly and the lack of data is a problem often encountered. In addition, the benefits brought by a sustainability perspective are only often perceived in the long-term, which makes its effective implementation difficult to promote.

Finally, life-cycle methodologies have been developed for the analysis of simple products. The application of such approaches to more complex systems, like a construction system, entails specific problems that need to be addressed in order to make them feasible [93].

This important issue has been addressed over the last years by some authors. An environmental input-output life-cycle assessment was proposed by Horvath and Hendrickson [98] for the comparison of a steel bridge and a steel-reinforced concrete bridge. Keoleian et al. [108] introduced a life-cycle environmental analysis for the comparative analysis of alternative decks. Other simplified life-cycle environmental approaches, restricted to one or two indicators, were also proposed by Itoh and Kitagawa [106], Lounis and Daigle [122] and Bouhaya et al. [64]. More recently, a life-cycle approach integrating environmental, economical and social criteria was proposed by Gervásio [93]. In addition, the proposed approach enables the consideration of uncertainties inherent to life-cycle analysis by means of a probabilistic analysis [92].

In terms of research projects, a project developed by the Nordic countries aimed to evaluate and compare different life-cycle issues of a general bridge design, [82]. This project addressed life-cycle costs, environmental impacts and cultural values.

4.2.2 *General framework for Life Cycle Environmental Analysis*

The framework for Life Cycle Environmental Analysis (LCA) adopted in this project is according to ISO standards 14040 [103] and 14044 [104]. These standards specify the general framework, principles and requirements for conducting and reporting life-cycle assessment studies. According to these standards, life-cycle assessment shall include (i) definition of goal and scope, (ii) inventory analysis, (iii) impact assessment, (iv) normalization and weighting, and (v) interpretation of results. The step of normalization and weighting is considered to be optional in ISO standards and will not be addressed in the life-cycle environmental analysis. Thus, the complete flowchart for the environmental life-cycle analysis is represented in Figure 4-2.

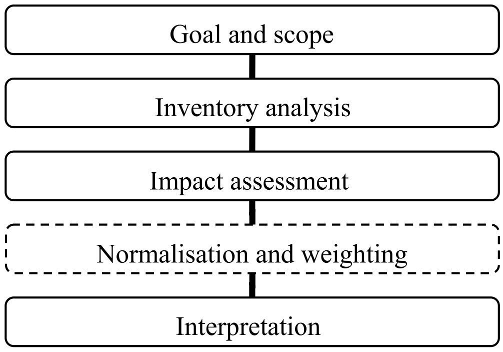


Figure 4-2. Scheme of the environmental life-cycle analysis.

In the following paragraphs, a brief description of the main steps of LCA is provided.

4.2.3 *Main steps of LCA*

4.2.3.1 Goal of the LCA

The goal of an LCA shall clearly state the intended application, the reasons for carrying out the analysis and the intended audience, i.e. to whom the results of the study are intended to be communicated [103].

In this case, the general goal of the LCA is to evaluate the environmental performance of composite motorway bridges over their life-cycle. The period of analysis is assumed to be 100 years. The life-cycle analysis will highlight main advantages and disadvantages of this kind of structures and will allow providing recommendations for further improvements.

4.2.3.2 Functional unit

The next step is the definition of the functional unit of the product system under analysis (e.g. a composite bridge).

A functional unit is a measure of the performance of the functional outputs of the product system [103]. The primary purpose of a functional unit is to provide a reference to which the inputs and outputs are related. This reference is necessary to ensure comparability of LCA results. Comparability of LCA results is particularly critical when different systems are being assessed to ensure that such comparisons are made on a common basis.

In the aim of this project the general definition of the functional unit is a motorway bridge, designed for a service life of 100 years, to support a dual-carriageway (for case studies A and C) or to overpass a dual-carriageway (for case studies B). This general definition was adopted in all the case studies that were performed throughout the project.

4.2.3.3 Scope of the LCA

The system boundaries determine which unit process shall be included within the LCA [103]. Several factors determine the system boundaries, including the intended application of the study, the assumptions made, cut-off criteria, data and cost constraints, and the intended audience.

The system boundary adopted in this project is represented in Figure 4-3. All stages over the complete life-cycle of the bridges, from raw material extraction until end-of-life procedures, are included. Furthermore, the transportation of materials and equipments are also within the system boundary.

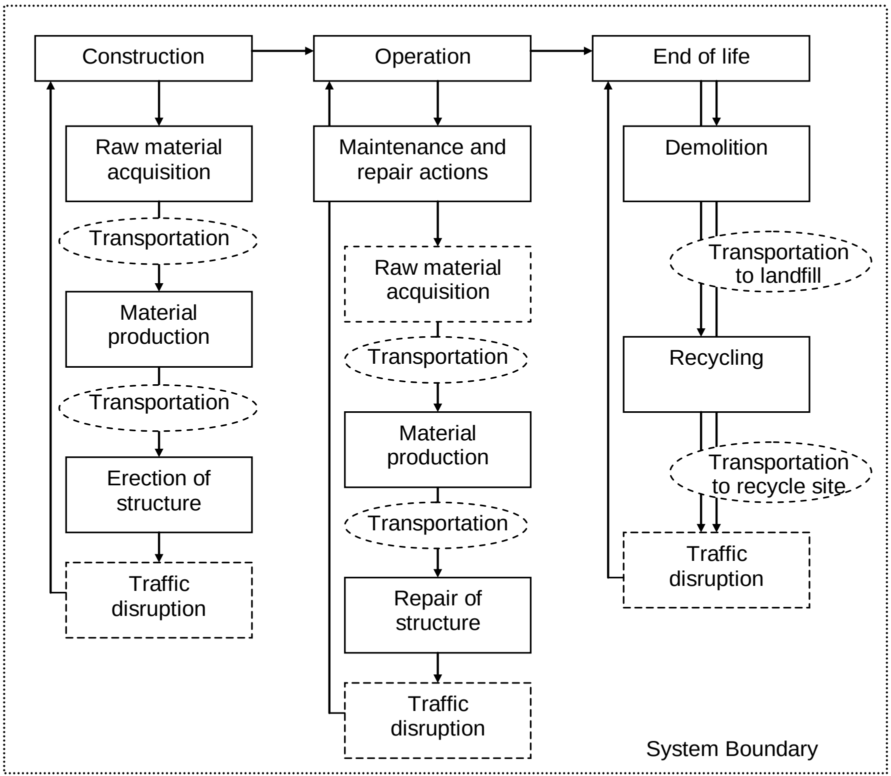


Figure 4-3. System boundary of the LCA [93].

When the composite bridge is built (assuming that the motorway is under service) or it goes under repair, traffic congestion results from delays over the construction work zone. This construction related delay results in additional fuel consumption and related emissions. The effects of traffic congestion were also taken into account in the LCA.

4.2.3.4 Methodology for impact assessment

The impact assessment stage of a LCA is aimed at evaluating the significance of potential environmental impacts using the results of the life-cycle inventory analysis. In general, this process involves associating inventory data with specific environmental impact categories, and is made in two parts (i) mandatory elements, such as selection of environmental indicators and classification; and (ii) optional elements, such as normalization, ranking, grouping and weighting.

The classification implies a previous selection of appropriate impact categories, according to the goal of the study, and the assignment of inventory results to the chosen impact categories. Characterization factors are then used representing the relative contribution of an inventory result (m_i) to the impact category indicator result, as expressed by the following expression (4-1):

$$impact_{cat} = \sum_i m_i \times charact\_factor_{cat,i} \qquad (4\text{-}1)$$

The environmental indicators adopted in the life-cycle approach are listed in Table 4-1.

Table 4-1. Environmental indicators for LCA

Indicator		Space scale	Time scale
Global Warming Potential	GWP	Global	100 yrs
Acidification Potential	AP	Local/continental	∞
Eutrophication Potential	EP	Local/continental	∞
Photo Ozone Creation Potential	POCP	Local/continental	-
Ozone Depletion Potential	ODP	Global	∞
Terrestrial Ecotoxicity Potential	TETP	Local/regional/continental/global	100 yrs
Human Toxicity Potential	HTP	Continental/global	100 yrs
Abiotic Depletion Potential	ADP	Global	-

All the indicators represented in Table 4-1 are evaluated according to expression (4-1), and they are further described in the following paragraphs. The characterization factors adopted in this approach are provided from the methodology developed by the Centre of Environmental Sciences [70] in the University of Leiden.

4.2.3.5 Indicators of environmental performance

4.2.3.5.1 Global Warming Potential (GWP)

The global warming indicator measures the impact of human emissions on the radiative forcing of the atmosphere.

GWPs are defined as the ratio of the time-integrated radiative forcing from the instantaneous release of 1 kg of a trace substance relative to that of 1 kg of a reference gas [101].

The generally accepted authority on GWPs is the Intergovernmental Panel on Climate Change (IPCC), and in their definition of GWPs the reference gas is carbon dioxide (CO_2). Therefore, the global warming potential of substance *i* (GWP_i) is given by the ratio between the increased infrared absorption due to the instantaneous emission of 1 kg of the substance and that due to an equal emission of carbon dioxide (CO_2), both integrated over time, as given by [101],

$$GWP_{T,i} = \frac{\int_0^T a_i\left[c_i(t)\right]dt}{\int_0^T a_{CO_2}\left[c_{CO_2}(t)\right]dt} \qquad (4\text{-}2)$$

where, a_i is the radiative forcing per unit concentration increase of greenhouse gas *i* (in $W.m^{-2}.kg^{-1}$), $c_i(t)$ is the concentration of greenhouse gas *i* at time *t* after the release (in $kg.m^{-3}$), *T* is the time over which integration is performed (in yr) and the corresponding quantities for the reference gas in the denominator.

GWPs were calculated for three time horizons of 20, 100 and 500 years. Table 4-2 indicates the GWPs of three of the most important greenhouse gases for the three time horizons.

Table 4-2. GWPs for given time horizons (IPCC, 2007) (in kg CO_2 eq./kg).

	20 years	100 years	500 years
Carbon Dioxide (CO_2)	1	1	1
Methane (CH_4)	62	23	7
Nitrous oxide (N_2O)	275	296	156

Hence, according to expression (4-1), the determination of the indicator "Global Warming" is given by,

$$Global\ \ Warming = \sum_i GWP_i \times m_i \qquad (4\text{-}3)$$

where, m_i is the mass of substance *i* released (in kg). This indicator is expressed in *kg of CO_2 equivalents*.

4.2.3.5.2 Ozone Depletion Potential (ODP)

An ozone depletion indicator is derived through several properties of a gas, which include its stability to reach the stratosphere and the amount of bromine or chlorine the gas carries. These properties are then compared to CFC-11 (although CFC-11 is now banned by the Montreal Protocol in industrialized nations, it is still manufactured in many developing economies). The properties of each gas are then compared to the properties of CFC-11 and converted into CFC-11 equivalents. Then the individual equivalents are added together for the overall ozone depletion indicator score, which represents the total quantity of ozone depleting gases released. The ozone depletion potential of substance *i* (ODP_i) is given by (4-4),

$$ODP_i = \frac{\delta[O_3]_i}{\delta[O_3]_{CFC-11}} \qquad (4\text{-}4)$$

where, $\delta[O_3]_i$ represents the change in the stratospheric ozone column *i* the equilibrium state due to annual emissions of substance i (in kg.yr^{-1}), and $\delta[O_3]_{CFC-11}$ the change in this column in the equilibrium state due to annual emissions of CFC-11.

OPDs were compiled by the World Meteorological Organisation (WMO). These OPDs are steady-state and they describe the integrated impact of an emission of a substance on the ozone layer compared with CFC-11 [70]. These ODPs are recommended for situations in which the time span of interest is eternity. In other cases, time-dependent ODPS are recommended [70]. Some of the OPDs are indicated in Table 4-3 for selected substances.

Table 4-3. Time-dependent OPDs for some substances [70] (in kg CFC-11 eq./kg)

	Steady-state (t ≈ ∞)	**40 years**	**30 years**	**25 years**	**20 years**	**15 years**	**10 years**	**5 years**
CFC-11	1	1	1	1	1	1	1	1
CFC-10	1.2	1.14	1.2	1.22	1.22	1.23	1.25	1.26
Halon 1211	5.1	7.1	8.0	8.5	9	9.7	10.5	11.3
Halon 1301	12.0	10.8	10.7	10.6	10.5	10.5	10.4	10.3

Thus, the determination of the indicator ozone depletion is given by,

$$Ozone\ \ Depletion = \sum_i ODP_i \times m_i \qquad (4\text{-}5)$$

where, m_i is the mass of substance *i* released (in kg). This indicator is expressed in *kg of CFC-11 equivalents*.

4.2.3.5.3 Photochemical Ozone Creation Potential (POCP)

Photo-oxidants may be formed in the troposphere under the influence of ultraviolet light, through photochemical oxidation of volatile organic compounds (VOCs) and carbon monoxide (CO) in the presence of nitrogen oxides (NO_x) [70]. This chemical reaction is "non-linear," meaning that sometimes the NO_x concentration will drive the reaction, and other times, it's the VOC that drive the reaction. Various indicators take low, average and high NO_x concentrations to calculate an overall score. Photochemical ozone creation potentials assess various emission scenarios for VOCs. Therefore, the photochemical ozone creation potential of a VOC (POCP) is given by the ratio between the change in ozone concentration due to a change in the emission of that VOC and the change in the ozone concentration due to a change in the emission of ethylene (C_2H_4) [70], as expressed by,

$$POCP_i = \frac{a_i / b_i}{a_{C_2H_4} / b_{C_2H_4}} \qquad (4\text{-}6)$$

where, a_i represents the change in ozone concentration due to a change in the emission of VOC i; b_i represents the integrated emission of VOC i, up to that time; and a_{C2H4} and b_{C2H4} the respective parameters for the reference substance, ethylene.

POCPs were calculated for two scenarios (i) a scenario with a relatively high background concentration of NO_x; and (ii) a scenario with a relatively low background concentration of NO_x. These two characterization factors are indicated in Table 4-4 for some selected substances.

Table 4-4. POCPs for different concentration of NOx and for some substances [70] (in kg C2H4 eq./kg).

	High-NO_x POCPs	**Low-NO_x POCPs**
Acetaldehyde (CH_3CHO)	0.641	0.200
Butane (C_4H_{10})	0.352	0.500
Carbon monoxide (CO)	0.027	0.040
Ethyne (C_2H_2)	0.085	0.400
Methane (CH_4)	0.006	0.007
Nitrogen oxide (NO_x)	0.028	no data
Propene (C_3H_6)	1.123	0.600
Sulphur oxide (SO_x)	0.048	no data
Toluene ($C_6H_5CH_3$)	0.637	0.500
Non-methane volatile organic compounds (NMVOCs)	0.150	0.150

Thus, the determination of the indicator Photo-oxidant formation is given by,

$$Photo-oxidant \quad formation = \sum_i POCP_i \times m_i \tag{4-7}$$

where, m_i is the mass of substance i released (in kg). This indicator is expressed in *kg of ethylene (C_2H_4) equivalents*.

4.2.3.5.4 Acidification Potential (AP)

Acidification in one of the impact categories in which local sensitivity plays an important role. The characterisation factors adopted in this work are based in the model RAINS-LCA, which takes fate, background depositions and effects into account [100]. Based in this model, Huijbregts [100] developed characterisation factors for 44 regions in Europe and average European factors, by a weighted summation of the regional factors for each acidifying emission, as given by the following expression [70]:

$$AP_{x,r} = \frac{\sum_{e \in Europe} A_{e \in j} \times \frac{t_{r,x,j}}{CL_{e \in j}}}{\sum_{e \in Europe} A_{e \in j} \times \frac{t_{r,ref,j}}{CL_{e \in j}}} \tag{4-8}$$

where, $AP_{x,r}$ is the regional acidification potential of substance x in region r, A_e represents the ecosystem e (in grid cell $_j$), $CL_{e\epsilon j}$ is the critical load for ecosystem e (in grid cell j), $t_{r,x,j}$ represents a transport factor (the fraction of $E_{r,x}$ deposited on j), and $E_{r,x}$ is the emission of substance x in region r.

The average European characterisation factors for acidification are represented in Table 4-5, and they are compared with the values calculated for Western and Eastern Europe [100]. The reference area is Switzerland.

Table 4-5. Acidification potentials (in kg Swiss SO2 eq.)[100].

	Ammonia (NH_3)	**Nitrogen Oxide (NO_x)**	**Sulfur Oxide (SO_2)**
Average Europe	1.60	0.50	1.20
West Europe	1.30	0.41	0.79
East Europe	1.80	0.70	1.60

Thus, the determination of the indicator acidification is given by,

$$Acidification = \sum_i AP_i \times m_i \tag{4-9}$$

where, m_i is the mass of substance *i* released (in kg). This indicator is expressed in *kg of SO_2 equivalents.*

4.2.3.5.5 Eutrophication Potential (EP)

The eutrophication indicator is given by the aggregation of the potential contribution of emissions of N, P and C (given in terms of chemical oxygen demand, COD) to biomass formation [70]. The Eutrophication Potential of substance *i* reflects its potential contribution to biomass formation and it's given by the relation,

$$EP_i = \frac{\nu_i / M_i}{\nu_{ref} / M_{ref}} \tag{4-10}$$

where, ν_i and ν_{ref} are the potential contributions to eutrophication of one mole of substance *i* and one mole of the reference substance, respectively; and M_i and M_{ref} are the mass of *i* and the mass of reference substance, which in this case is $^{-}PO_4^{3-}$. Some characterization factors for selected substances are indicated in Table 4-6.

Table 4-6. Eutrophication potentials (in kg PO_4^{3-} eq./kg) [70].

	Ammonia (NH_3)	**Nitrogen Oxide (NO_x)**	**Nitrate (N)**	**Phosphate (P)**
EPi	0.35	0.13	0.10	1.00

The eutrophication indicator is given by,

$$Eutrohication = \sum_i EP_i \times m_i \tag{4-11}$$

where, m_i (kg) is the mass of substance *i* released to the air, water or soil; and EP_i is the Eutrophication Potential of the substance *i,* expressed in *kg* PO_4^{3-} *equivalents*.

4.2.3.5.6 Human Toxicity Potential (HTP)

The indicator human toxicity covers the impacts on human health of toxic substances present in the environment.

The calculation of human toxicity potential is based in the model USES-LCA. This model is a nested model with five spatial scales: regional, continental and global (global is tripartite to reflect the arctic, temperate and tropical climate zones of the Northern hemisphere) [70]. To calculate a single characterisation factor for each emission compartment, the four factors calculated at the global and continental scales are aggregated on a population basis: the larger the exposed population, the greater the weight of the associated factor. Hence, the Human Toxicity Potential (*HTP*) of substance *i* emitted to emission compartment *ecomp*, is given by [70],

$$HTP_{i,ecomp} = \frac{\sum_r \sum_s PDI_{i,ecomp,r,s} \times E_{i,r} \times N_s}{\sum_r \sum_s PDI_{1.4-dichlorobenzene,air,r,s} \times E_{1.4-dichlorobenzene,r} \times N_s} \qquad (4\text{-}12)$$

where, N_s is the population density at scale *s*, $PDI_{i,ecomp,r,s}$ is the Predicted Daily Intake via exposure route *r*, at scale *s*, for substance *i* emitted to emission compartment *ecomp* (in day^{-1}), and $E_{i,r}$ is the effect factor, representing the human-toxic impact of substance *i*, (in day). In this model the reference substance is 1.4 dichlorobenzene. Thus, $PDI_{1.4\text{-}dichlorobenzene,r,s}$ is the Predicted Daily Intake resulting from the emission of 1000 kg of 1.4 dichlorobenzene to air (in day^{-1}), and $E_{1.4\text{-}dichlorobenzene,r}$ represents the human-toxic impact of 1.4 dichlorobenzene (in day).

The indicator result for human toxicity for a specific time horizon *t* can be calculated using the following expression [70]:

$$Human\ \ toxicity\ _t = \sum_i \sum_{ecomp} m_{i,ecomp} \times HTP_{i,ecomp,t} \qquad (4\text{-}13)$$

where, $HTP_{i,ecomp,t}$ is the human toxicity potential of substance *i* emitted to emission compartment *ecomp* for time horizon *t*, and $m_{i,ecomp}$ is the emission of substance *i* to compartment *ecomp* (kg).

The human toxicity potentials adopted in this project take into account the time horizon of 100 years. However, Table 4-7, indicates for some selected air emissions human toxicity potentials considering 20, 100, 500 years and infinity, at the global scale [100].

Table 4-7. Human toxicity potentials for different time horizons and global scale (in kg 1,4-DB eq./kg).

	20 years	**100 years**	**500 years**	**∞**
Ammonia (NH_3)	0.10	0.10	0.10	0.10
Barium (Ba)	172	175	181	756
Cadmium (Cd)	1.45E5	1.45E5	1.45E5	1.45E5
Cobalt (Co)	1.72E4	1.73E4	1.74E4	1.75E4
Lead (Pb)	24.2	29.1	48.2	467
Mercury (Hg)	212	264	416	6010
Nitrogen oxide (NO_x)	1.20	1.20	1.20	1.20
Particulates (PM)	0.82	0.82	0.82	0.82
Sulphur oxide (SO_x)	0.096	0.096	0.096	0.096

4.2.3.5.7 Terrestrial Ecotoxicity Potential (TETP)

This impact category covers the impacts of toxic substances on aquatic, terrestrial and sediment ecosystems.

The model adopted in this work for the calculation of ecotoxicity potentials is the same model as the one used for human toxicity potentials, which was described in the previous sub-section. Moreover, this model was used to calculate five subcategories of ecotoxicity potentials: freshwater aquatic, marine aquatic, freshwater sediment, marine sediment and terrestrial; each of them for different time horizons [100]. However, in this work, only terrestrial ecotoxicity is taken into account, in order to avoid giving undue weight to the category of ecotoxicity. In this case, the Terrestrial Ecotoxicity Potential (*TETP*) of substance *i* emitted to emission compartment *ecomp*, is given by [70]:

$$TETP_{i,ecomp} = \frac{PEC_{i,ecomp,terrestrial} \times E_{i,terrestrial}}{PEC_{1.4-dichlorobenzene,terrestrial,terrestrial} \times E_{1.4-dichlorobenzene,terrestrial}} \qquad (4\text{-}14)$$

where, $PEC_{i,ecomp,terrestrial}$ is the predicted concentration of substance *i* on earth due to the emission to compartment *ecomp*, $PEC_{1.4\text{-}dichlorobenzene,terrestrial}$ is the same predicted concentration but for 1.4-dichlorobenzene; $E_{i,r}$ is the effect factor, representing the toxic impact of substance *i* on terrestrial ecosystems and $E_{1.4\text{-}dichlorobenzene,terrestrial}$ is the same factor but for 1.4-dichlorobenzene . The indicator result for terrestrial ecotoxicity is calculated using the following expression:

$$terrestrial \quad ecotoxicity = \sum_{i} \sum_{ecomp} m_{i,ecomp} \times TETP_{i,ecomp} \qquad (4\text{-}15)$$

where $m_{i,ecomp}$ is the emission of substance i to compartment *ecomp*.

The terrestrial ecotoxicity potentials, adopted in this work for time horizons of 20, 100 and 500 years and infinity, at the global scale [100], are represented in Table 4-8 for some selected air emissions:

Table 4-8. Terretrial ecotoxicity potentials for different time horizons and global scale (in kg 1,4-DB eq./kg).

	20 years	**100 years**	**500 years**	**∞**
Barium (Ba)	0.359	1.55	4.12	4.86
Cadmium (Cd)	2.48	11.7	43.6	81.2
Cobalt (Co)	11.9	47.6	102	109
Lead (Pb)	0.0288	0.144	0.707	15.7
Mercury (Hg)	669	3170	12300	28300

4.2.3.5.8 Abiotic Depletion Potential (ADP)

The indicator abiotic depletion aims to evaluate the environmental problem related to the decreasing availability of natural resources. By natural resources it is understood the minerals and materials found in the earth, sea, or atmosphere and biota, that have not yet been industrially processed [95].

The model [95] adopted for abiotic depletion in this work, assumes that ultimate reserves and extraction rates together are the best way to represent the seriousness of resource depletion. This model is a global model based on ultimate reserves in the world combined with yearly depletion on a world level. Therefore, the abiotic depletion potential of resource i (ADP_i) is given by the ratio between the quantity of resource extracted and the recoverable reserves of that resource, expressed in kg of a reference resource, as given by expression (4-16):

$$ADP_i = \frac{DR_i}{(R_i)^2} \times \frac{(R_{ref})^2}{DR_{ref}} \qquad (4\text{-}16)$$

where, DR_i is the extraction rate of resource i (in kg/yr), R_i is the ultimate reserve of resource i (in kg), R_{ref} is the ultimate reserve of the reference resource (in kg), and DR_{ref} is the extraction rate of R_{ref} (in kg/yr). The reference resource is antimony.

Thus, the determination of the indicator abiotic depletion is given by,

$$Abiotic \quad Depletion = \sum_{i} ADP_i \times m_i \qquad (4\text{-}17)$$

where, m_i is the quantity of resource i extracted (in kg). This indicator is expressed *in kg of antimony* (the reference resource).

The abiotic depletion potentials developed from this model and adopted in this work are represented in Table 4-9 for some selected resources.

The abiotic depletion indicator as described in the previous paragraphs is recommended in the Guide [70] for LCA. This indicator is one of the most controversial indicators in LCA, and currently there’s no general consensus about the best indicator to describe this environmental problem (even within standardization bodies like CEN and ISO); therefore, it was decided to adopt this approach in the analysis. However, currently there are not sufficiently production and reserve data for some common abiotic resources, which could lead to some problems in the impact assessment stage of some inventory data.

Table 4-9. Abiotic depletion potentials for some resources (in Sb eq./kg).

Resource	ADs
Aluminium (Al)	1.00E-08
Calcite ($CaCO_3$)	2.83E-10
Coal	1.34E-02
Copper (Cu)	1.94E-03
Gas natural	1.87E-02
Iron ore (Fe)	4.80E-08
Lead (Pb)	1.35E-02

4.2.4 *Conclusions of LCA*

In the context of sustainability, the design of a bridge comprehends the life-cycle analysis of the bridge, from raw material acquisition to the bridge's decommissioning, integrating non-traditional aspects of design such as environment, economy and society.

For the life-cycle environmental analysis, it was decided to follow the framework for Life Cycle Environmental Analysis established in ISO standards 14040 [103] and 14044 [104]. Therefore, in this section, the environmental indicators selected for the life-cycle approach were described and the calculation methods for their calculation were introduced.

4.3 Life-cycle Costs

4.3.1 *Introduction*

Construction is a major industry throughout the world accounting for a sizeable proportion of most countries' Gross Domestic Product (GDP) and Gross National Product (GNP). The importance of the construction sector is not only related to its size but also to its role in economic growth [118]. Indeed, the construction industry accounts for about 10% of the GDP, which is the largest industrial sector in EU. This sector represents a quarter of the total output and involves 2.5 million of enterprises and 13 million employees [83]. Concerning bridges, surveys in Europe show that the value of bridges on the national network is estimated at 12 billion Euros in France, 23 billion Euros in the UK, 4.1 billion Euros in Spain and 30 billion Euros for Germany [68]. Besides, the operation phase with inspection / maintenance / rehabilitation may represent a significant portion of the total life-cycle costs (LCC) of a structure. Surveys in BRIME project show that the annual expenditure on maintenance and repair on national bridges is around 180 million Euros in England, 50 million Euros in France, 30 million Euros in Norway and 13 million Euros in Spain [68].

When designing civil engineering systems, the system performance must be considered as time-dependent [63]. Design approach should, consequently, take into account the desired performance not only at the initial stage but during the expected life-cycle of structures. If structural design generally aims at optimizing costs at the construction stage, the total life-cycle costs include not only construction costs but also other costs such as inspection, operation, end-of-life and user costs which may represent a significant portion of the total life-cycle costs of a structure [87]. This is why the traditional design concepts are now put into question to make a shift to a life-cycle level, which enables to create a more comprehensive view of the various costs that occur in the different phases of the structure service life.

Life-cycle costs (LCC) methodology is an economic evaluation method that takes account of all relevant costs over the defined time horizon (period of study), including adjusting for the time value of money. This procedure goes from the element costs to the structure costs and ends with the total life-cycle costs. While several lists of cost categorisations exist, such as "use, ownership, administration, engineering, manufacturing, distribution, service, sales, refurbishment costs", the detailed costs considered should depend on each project, system or product under consideration [141].

LCC concepts have been investigated these last years in different studies and research activities. Among others, [69], [97], [131], [134], [135], [138] and [89] have proposed LCC-based frameworks and provided some information on cost data. Since 2001, the European Union (EU) launched a Sustainable Development (SD) strategy, aimed at "dovetailing policies for economically, socially and ecologically sustainable development", [84], [118]. In particular, the Action C25 "Sustainability of Constructions: Integrated Approach to Life-time Structural Engineering", [65], was launched in 2006 within the COST framework, that is an intergovernmental framework for European Cooperation in Science and Technology, allowing the coordination of nationally-funded research on a European level. The Action C25 focused on the problem of "Sustainability of Constructions" which refers to the combination of methods of structural engineering with those of environmental impact assessment and life-cycle economy. Despite these recent efforts, the adoption and application of LCC might remain limited due to the practitioners' "imperfect understanding" [94].

The ISO 15686-5 methodology [105] defines the life-cycle costing as a technique which enables systematic economic evaluation of the life-cycle costs over the period of analysis, as defined in the agreed scope, Figure 4-4. ISO 15686-5 [105] also refers to whole life costing which is presented as a "methodology for the systematic economic consideration of all whole life costs and benefits over a period of analysis, as defined in the agreed scope". In a whole life costing approach, the projected costs or benefits may include finance, business costs, income from land sale and user costs.

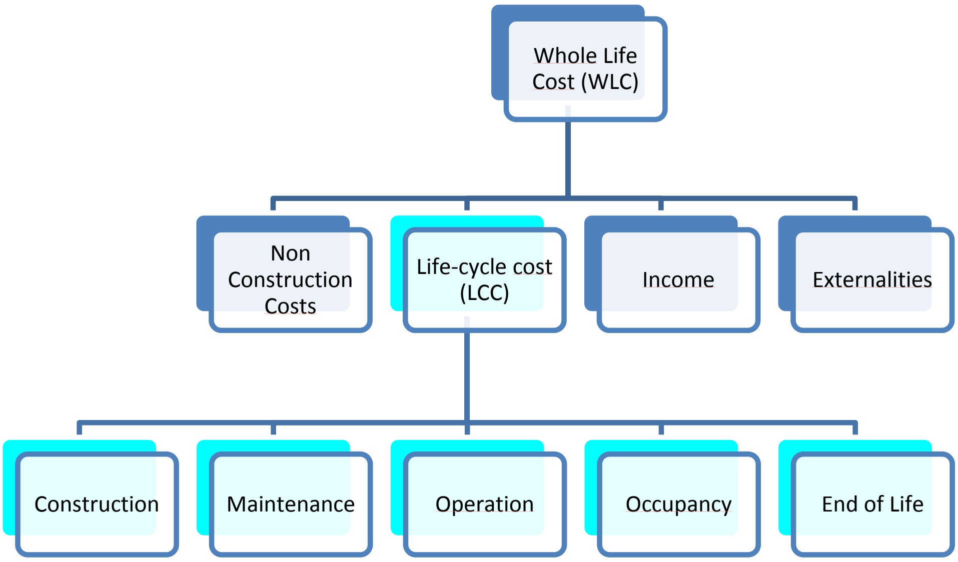


Figure 4-4. Whole Life cost and Life-cycle cost concepts [105].

LCC analysis methodology needs to cover not only the initial cost, but also the expected maintenance and rehabilitation costs over the life span of the structure, Figure 4-5. This leads to an expression of the LCC cost as follows (4-18)

$$C = C_c + C_o + C_d \tag{4-18}$$

where C_c = costs from the initial construction stage, C_o = costs from the operation stage, and C_d = costs from dismantlement event.

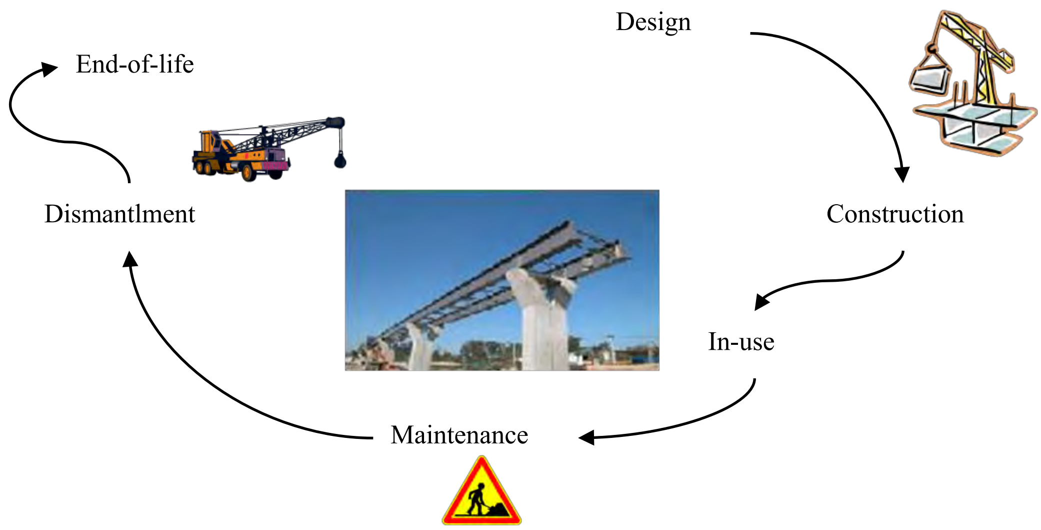


Figure 4-5. Life-cycle stages/costs from design to bridge end-of-life.

Selecting materials and components based only on initial costs may lead to an underestimate of future costs over the intended life of the system such as maintenance, repair, and reconstruction. In other words, the selection of materials and components based on a life-cycle costs analysis can significantly decrease the lifetime costs of construction, maintenance and repair, [31], [16], [107]. Similar conclusions can be drawn for dimensions of the structures, [119], [120]. In this context, one important motivation to use life-cycle cost analysis (LCCA) is to balance the decrease of operation and maintenance costs with a possible increase of initial costs, [109]. By considering all these costs in the decision process and ensuring performance constraints are satisfied, solutions that may be more expensive than others at the construction stage can finally be more attractive when considering the overall life service of the structure, see Figure 4-6.

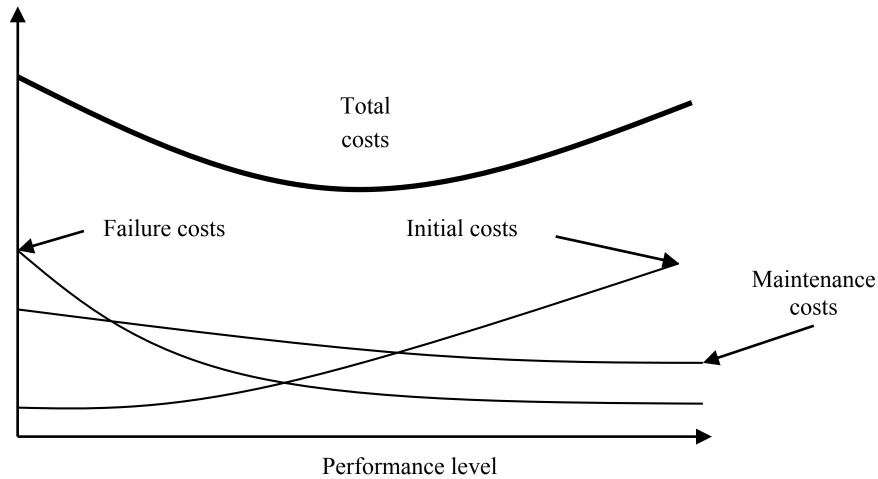


Figure 4-6. Schematic representation of life-cycle costs (adapted from [118]and [88]).

4.3.2 *Economic evaluation method for LCC*

The costs included in LCC analysis being incurred at varying points in time, there is a need to convert them into a value at a common point in time [136]. Several methods exist to lead LCC among which:

- the payback method, which determines the time required to return to the initial investment,
- the equivalent annual costs, which expresses the costs per year of owning and operating an asset over its entire lifespan,

– the internal rate of return, which is the discount rate at which the net present value of costs (negative cash flows) of the investment equals the net present value of the benefits (positive cash flows) of the investments,
– the net present value approach which directly applies discount factors to each year projected cash flow.

The net present value approach mentioned above is one of the most used methods to compare past and future cash flows with those of today. To make costs time-equivalent, the approach discounts them to a common point in time, the discount rate of money reflecting the investor's opportunity costs of money over time. The net present value can be calculated as follows:

$$NPV = \sum_{k=1}^{N} \frac{C_k}{(1+r)^k} \tag{4-19}$$

where:

NPV are the life-cycle costs expressed as a present value,

k is the year considered,

C_k is the sum of all cash flows in year n,

r is the discount rate,

N is the number of actions to be considered during the service lifetime.

The yearly profile of 1 unity of money is shown for illustration in Figure 4-7. It is noted that a steep drop of the discounted costs is observed for high discount rate values. Also, it is shown that choosing r = 6 or 8% leads to a monetary value close to zero after sixty years. According to [126], the American FHWA (Federal Highway Administration) recommends to keep the real discount rate within the range of 3 to 5 %. [68] and [136] in Sweden suggest the value of 4 % to be used for the discount rate. Cremona [72] argues that discount rates more than 5% are generally considered as too high when comparing costs over a long-time period and a yearly discount rate close to 2 or 3% is generally considered as a more realistic value.

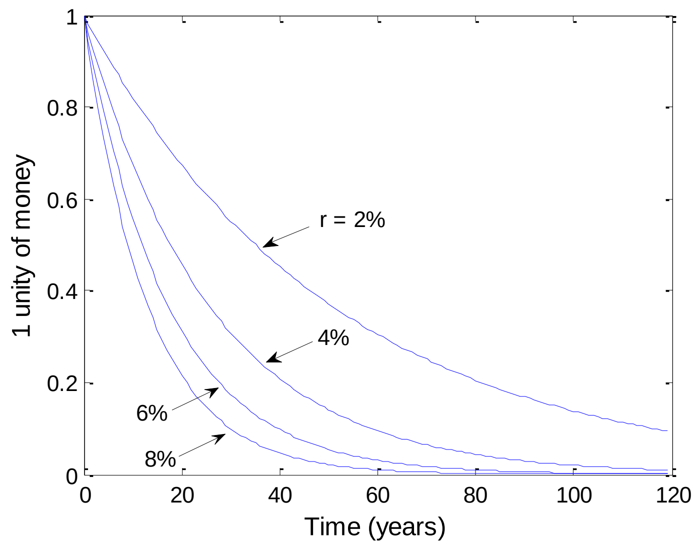


Figure 4-7. Profile of one unity of money for different values of r.

Obviously, the value of the yearly discount rate used is crucial since the current worth of money (*NVP*) is highly sensitive to this parameter. Indeed, the higher the discount rate, the more importance is given to the near-present. Choosing a high discount rate may then promote management strategies with low initial costs and a costly end-of-life. It is noted that a discount rate fixed at 0% has already been considered in Japan for some projects, [72]. Therefore, the choice of the discount rate is delicate and has to be in agreement with the time horizon. It is noted that the discount rate is fixed at 2% in the LCCA performed in the project SBRI for a 100-year service life.

4.3.3 *Applications*

As mentioned in section 4.3.1 and in comparison with current approaches, which generally estimate only the direct costs for construction and maintenance, LCC extends the analysis over the whole life of the project, showing the real value of the investment. Such an analysis investigates the costs related to the entire life-cycle in combination with the assessment of structural performances over time. Initial costs (design, material production, and fabrication), operation costs (inspection and repair costs) and end-of-life costs are then assessed, Figure 4-8.

It is mentioned that the costs of failure (which comprises costs associated with structural failure multiplied by their probability of occurrence) is not investigated in this project, which focuses only on normal operation scenarios during the service life. The application of LCC to steel-concrete composite bridges, according to the construction/operation/end-of-life scheme is detailed in the following of this section.

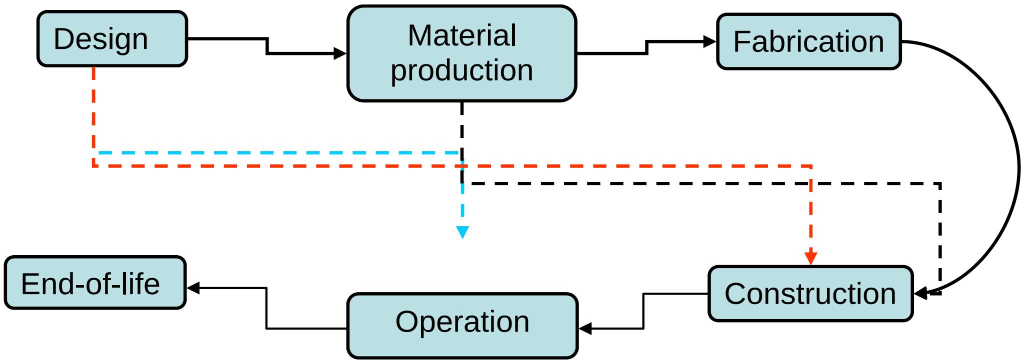


Figure 4-8. From design to end-of-life costs.

4.3.3.1 Construction

Expenses associated with steel-concrete composite bridge construction mainly include costs for (i) foundation, (ii) substructure with abutments, piles and bearings, (iii) superstructure with steel girder/box (for composite bridge), concrete deck and equipments (expansion joints, road surface, waterproofing layer, metal cornice gutter, railing and protection). It is noted that these costs should include all materials and work costs needed for each component. Obviously, the different design solutions of a composite bridge are associated with different construction costs according to the type of materials used and the fabrication/erection process.

4.3.3.2 Operation stage

4.3.3.2.1 Inspection

All structures have to be inspected and maintained. In particular, bridge inspections are essential for determination of intervention strategies. The time intervals between these measures depend on the type of bridge, the experience in the different countries, the economic resources available, the average daily traffic value, the usage of de-icing salt and so on. Also, inspection strategies (intensities and frequencies of inspections) may be different in each country based on climate conditions and prioritization strategies proper to each country, [141]. Three basic types of inspection are detailed below:

- *routine inspections (regular):* performed with short intervals, the goal being to detect major defects (e.g. damaged safety barrier or broken drainage system);
- *major inspections:* visual inspection of all parts of bridge structure, the aim being to check the condition of all parts of the bridge and to make a global visual assessment of the bridge condition;

– *special inspections:* carried out when there is a particular problem or cause for concern either found during an inspection or already discovered on other similar bridges. Special inspections may also be carried out after the occurrence of extreme events such as flooding, and earthquakes.

For further information compare also section 2.3 and [3].

4.3.3.2.2 Maintenance

During the operation stage of the bridge, the estimated maintenance activities are taken into account, the objective being that the performance of the bridge (associated with serviceability and safety concepts) always remains above a minimum threshold, Figure 4-9. This point corresponds to the end of the service life if no other rehabilitation action is conducted.

A regular interval between interventions is generally considered by highway agencies to assess the costs in the LCC analysis. For example, Irzik et al. [2] estimate future times at which maintenance actions/rehabilitation will be performed based on the average service life of elements of the bridge. It is noted that intervals are updated in this model, based on the measures that are performed on the bridge. Figure 4-9 illustrates the link between the life-cycle performance and the life-cycle costs.

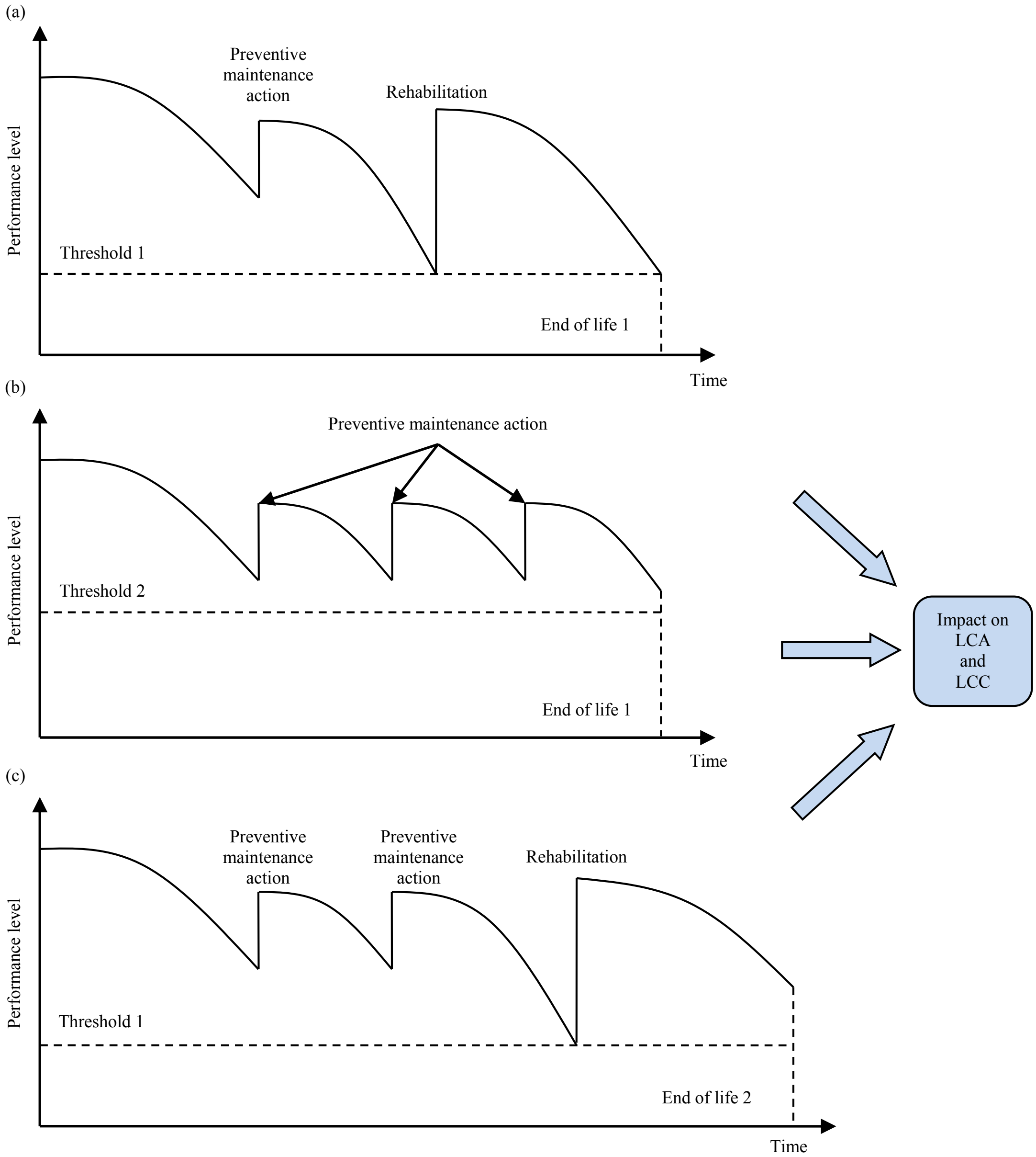


Figure 4-9. Bridge performance profiles in link with time horizon and performance thresholds.

Depending on the minimum allowable performance threshold, preventive, essential and rehabilitation actions might be decided in a different way (compare cases a and b in Figure 4-9). Besides, the definition of the time horizon might be crucial to orientate the choice to be made between different maintenance strategies (e.g., compare cases a or b with c in Figure 4-9). Many researchers and practitioners [81], [140], [124] have proposed optimal maintenance strategies for critical structural elements. In particular, Ferry and Flanagan [85] show that LCC analysis can be used as a management tool throughout the service life of the structure (for inspection, maintenance/rehabilitation and dismantlement) whenever different options are available, to determine the lowest LCC.

For further information compare also section 2.3 and [4].

4.3.3.1 End-of-life

In the end-of-life stage, it is assumed that the bridge is demolished and that the materials are sorted in the same place before being sent to their final destination. For steel-composite bridges, it is assumed that the steel structure is going to be reused. The remaining parts, which are generally concrete and bitumen materials, are cut down and transported to waste disposal areas. In this context, end-of-life costs should take into account the costs of bridge dismantlement (labor work, equipments, road warning signage), costs of transportation and costs for deposition of materials and/or revenue due to recycling of materials.

4.3.4 *Conclusions of LCC*

The total life-cycle costs include not only construction costs but also other costs associated with inspections, operation/maintenance actions and end-of-life. Service-life (operation, maintenance) and end-of-life costs may represent a significant portion of the total life-cycle cost of a structure. This is why the traditional design concepts are now put into question to make a shift to a life-cycle level and optimize life-cycle costs [16]. One important motivation to use a life-cycle approach is to balance the decrease of operation and maintenance costs with a possible increase of initial costs. In other words, a solution more expensive at the construction stage can finally be more attractive when considering the overall service life of the structure. Such an analysis needs to enable accurate comparative cost assessments over a specified period of time, taking into account all relevant economic factors from initial costs to future operation and maintenance costs until the destruction of the structure.

4.4 User costs

Contrary to the owner costs that are direct measurable costs, the user costs are indirect and hardly measurable. In the case of highway bridges, these costs are those incurred by the users due to maintenance operations of highway structure causing congestion or disruption of the normal traffic flow. These costs are not directly measurable but the traffic delays that lead to them can be measured. Traffic delay costs have, consequently, to be predicted on the basis of estimated delay and vehicle operation costs which include additional costs of fuel plus additional costs of vehicle maintenance. These costs are briefly described below:

- *traffic delay costs* result from an increase in travel time through the work zone due to speed reductions, congestion delays or increased distances as a result of detour. This costs are influenced by many factors such as current and future traffic, bridge capacity, the timing, duration, and frequency of work-zone-induced capacity restrictions, and the unit costs for delay.

- *vehicle operating costs* are due to the level of service loss caused by the maintenance operations on highway structures. The disruption of normal traffic causes speed reductions, increase of fuel and oil consumptions, tire wear and vehicle maintenance. In particular, additional costs of fuel are due to the fact that its consumption is significantly higher in congested conditions. Besides, vehicle maintenance costs increase since these items need faster replacement for vehicles travelling in congested conditions. Finally, the traffic disruption induced by maintenance works has a negative impact on road safety and consequently increases the accident rate on the road part affected by the works.

- accident costs due to an increase in the risk of accidents, healthcare and deaths when the traffic is disturbed during work periods on the bridge.

The current or future average daily traffic (ADT expressed in vehicles/day), based on the desired construction year, should be obtained from the traffic monitoring section. Due to factors such as population growth and economic prosperity, the volume of traffic on bridge may increase each year and can be estimated by (4-20):

$$ADT_t = ADT \times (1 + r_{tg})^{year_t - year_0} \qquad \textit{(4-20)}$$

where:

ADT_t is the average daily traffic to be used in the analysis at year t (vehicles/day),

$r_{tg}$is the expected traffic growth rate,

$Year_t$ is the year in which the ADT is calculated,

$Year_0$ is the year in which the ADT is measured.

User costs represent a critical aspect in the life-cycle of the structure, and must be considered along with environmental (section 4.2) and economic (section 4.3) aspects when making decisions during the overall service life. In particular, user costs enable to quantify, in economic terms, the potential safety and mobility benefits of functional improvements to bridges. In the SBRI-project, a user cost model QUEWZ-98 ([62], [71], [99], [110]) is used to estimate the differences in terms of user costs between different design solutions [16].

5 LIFE-CYCLE PERFORMANCE

5.1 General

The life-cycle performance of each bridge is mainly described by the performance of critical details. Therefore a good knowledge of the behaviour of the details during the entire lifespan of a bridge is essential for holistic analyses. Degradation can be divided into several processes. For bridges, fatigue, corrosion and carbonation are the processes to be looked at, see Figure 5-1.

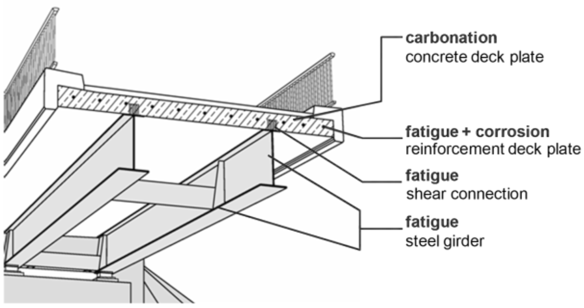


Figure 5-1. Degradation processes.

A scheduling of inspections and maintenance actions should be done based on a detailed description of the life-cycle performance of the affected details. Thus, life-cycle costs and emissions can be reduced. Intervals of bridge inspections can also be optimized by the knowledge of the adequate non-destructive testing methods to early detect defects. A comparison of these methods was done during fatigue tests on a standard detail of steel girders, the transverse stiffener. In further fatigue tests the influence of cracked concrete to the behaviour of horizontally lying shear studs was analysed. As fatigue can lead to a serious decrease of structural safety and is a central problem in bridges, focus was laid on the analyses of two details affected by fatigue.

Whereas for safety fatigue is of importance, corrosion can be dealt without comprising safety and is based on a well-established knowledge for the protection of steel surfaces. However for steel-composite bridges corrosion may occur between the concrete deck and the steel girder and may create a serious problem since renewal is difficult. Additional problems may be caused when a sealant is used and corrosion attacks the steel behind the sealant. Therefore prototype testing in a test chamber led to a detailed description of corrosion processes and to recommendations.

Carbonation processes impact the reinforced concrete structure of composite bridges leading to degradation. Several models can be applied to assess carbonation. Here models were compared and methods to monitor carbonation analysed. No specific development is considered in the frame of this project, but description is based on existing knowledge.

Knowing and being able to describe the life-cycle under deterioration processes of bridge details gives the possibility to optimize the structure in regard of sustainable aspects.

In the following, detailed investigations on the degradation processes fatigue, corrosion and carbonation are described.

5.2 Fatigue behaviour

5.2.1 *Introduction*

Bridges are structures which are subjected to traffic loading over a long lifespan. Traffic loading is recurrent and beyond that, due to increasing traffic volume, it must be calculated with an increased loading till the end of the life-cycle. Fatigue therefore cannot be neglected in bridges and is a main degradation process affecting the structure. In steel-composite bridges various details can be affected by fatigue and cause cracking. A classification by the severity of the induced damage can lead to the identification of the most critical details during the life-cycle. A performed literature review showed that critical spots are not only located in steel but also in composite details.

The transverse stiffener was selected as a typical detail in steel. Transverse stiffeners not only are used in steel bridges but also in composite bridges and are a common detail which faces crack problems due to fatigue. Post-weld treatment by high frequent hammering was used to improve that crucial resistance. An effective shear connection in composite bridges are horizontally lying shear studs in combination with an omitted steel flange. Also this detail was investigated, Figure 5-2. The existing degradation models for these details were analysed and improved by results from own prototype tests in order to be included in the overall assessment.

Figure 5-2.Details analysed.

The detail of transverse stiffeners welded by fillet welds to the flange is a typical detail which may be critical for steel and steel-composite bridges in regard of fatigue. Not only are transverse stiffeners used in steel bridges but also in composite bridges and are a common detail which may initiate crack problems of the flange due to fatigue. Post-weld treatment by high frequent hammering was used to improve that crucial resistance. Intervals of bridge inspections can be optimized by the knowledge of the adequate non-destructive testing methods to early detect defects. To establish a certain inspection strategy it is crucial to know the abilities of non-destructive testing methods detecting fatigue cracks at certain time and to a certain extend in order to specify the appropriate method and the necessary inspection intervals.

An effective shear connection in composite bridges are horizontally lying shear studs in combination with an omitted steel flange, e.g. for prefabricated composite bridge girders or connections of the concrete slab to the outer main girder in typical arch bridges. The existing degradation models for these details were analysed and improved by results from own prototype tests in order to be included in the overall assessment.

5.2.2 *Fatigue behaviour of transverse stiffeners*

5.2.2.1 Description of the detail

The knowledge of the fatigue behaviour of the detail of transverse stiffeners is essential for describing the life-cycle of steel and composite bridges. Focus here was laid to optimize this detail in regard of a long-living bridge structure. Models with respect to fatigue can be either based on fracture mechanics or on damage based approach. The rule of Palmgren-Miner is applied in the damage based approach. A linear accumulation of stress ranges $\Delta\sigma$ over number of cycles N is performed with e.g. the rainflow method. In logarithmic S-N curves the fatigue performance for a detail can be shown graphically. At two million cycles the stress range of the detail category is given, the so called classification value. This stress range must be known for an application of this fatigue approach. The existing knowledge of describing fatigue in bridge design is mainly based on this damage based approach. Once a S-N curve can be plotted for a detail the fatigue degradation process can be described according to Palmgren-Miner. In Table 8.4 of EN 1993-1-9 [77] the transverse stiffener is given as detail number 7 with the detail category of 80. This means that with an exceeding probability of 95% a failure of the detail occurs at 2,000,000 cycles N when loaded with a stress range $\Delta\sigma_C$ of 80 N/mm².

The detail category is mainly based on various tests recorded by data sheets in the commentary to EN 1993-1-9. For a reliable statistical evaluation it is important to analyze a significant amount of data. Therefore three sets of tests (identification codes 3_4_NLC16, 3_4_NLC18 and 3_4_NLC18 in the commentary to EN 1993-1-9) were chosen, giving reference to 95 tests of the year 1974 on this detail. For these 95 tests the parameters of the S-N curve, as linear regression curve, were determined. At N=2,000,000 cycles a medium stress range of $\Delta\sigma_{50\%}$ =114 N/mm² can be calculated for m=2.84 as a variable slope according to statistic analyses. S-N curves in standards as EN 1993-1-9 are usually plotted with a constant slope of m=3 for welded details which leads to $\Delta\sigma_{50\%}$ =116 N/mm² on the basis of the test data, corresponding to the 95%-fractile of the detail category in EN 1993-1-9.

5.2.2.2 Fatigue tests on transverse stiffeners with application of post-weld treatment and non-destructive testing

In a test series of 4 girders the fatigue behaviour of transverse stiffeners is compared to existing data in the commentary to EN 1993-1-9 [77]. An optimization of this detail in regard of a long-living structure was to be achieved. To extend the service life of welded details under fatigue loading promising results have been gained by [73], [129] with the application of ultrasonic impact treatment (high frequent hammering) as post-weld treatment method. Therefore, the lifetime of the welded detail is investigated in two tests applying post-weld treatment. By this high frequent hammering on one hand the geometry at the weld transition is improved and on the other hand the notch effect is reduced by introduction of compressive stresses at the surface.

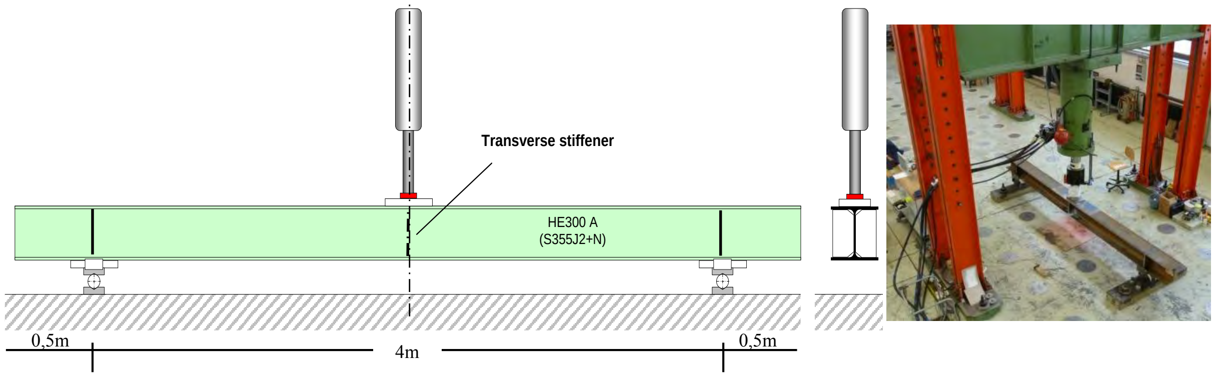


Figure 5-3. Test set-up of girder tests.

The detail of the transverse stiffeners was tested in girders with a cross-section HE 300 A and of 5 m total length. The set-up of the steel girders spanning 4 meters, as shown in Figure 5-3, was used for the fatigue tests. The same stress range of $\Delta\sigma = 187$ N/mm² was introduced at the location of the welds of the transverse stiffeners and allows for comparability between the girders. The end of the fatigue test, and therefore the failure of the girder, was defined as the complete breakthrough of a girder flange.

For the standard girders (G1 and G2) a number of cycles N = 500,000 was calculated and post-weld treatment by high frequent hammering was expected to double the number of cycles (G3 and G4). With N2= 520,200 and N1= 702,000 cycles the test reproduced well the tests found in the commentary to EN1993-1-9 (3_4_NCL 16 – 18). The weld of G3 was improved by high frequent hammering, here pneumatic impact treatment was used [91], lead to a breakthrough of the flange after N3= 1,929,000. After N4= 4,500,000 cycles the test of G4 had to be stopped without any fatigue failure occurring at the transverse stiffeners. Figure 5-4 illustrates the S-N curve of the transverse stiffener and the improved fatigue behaviour of G3 and G4 due to the post-weld treatment.

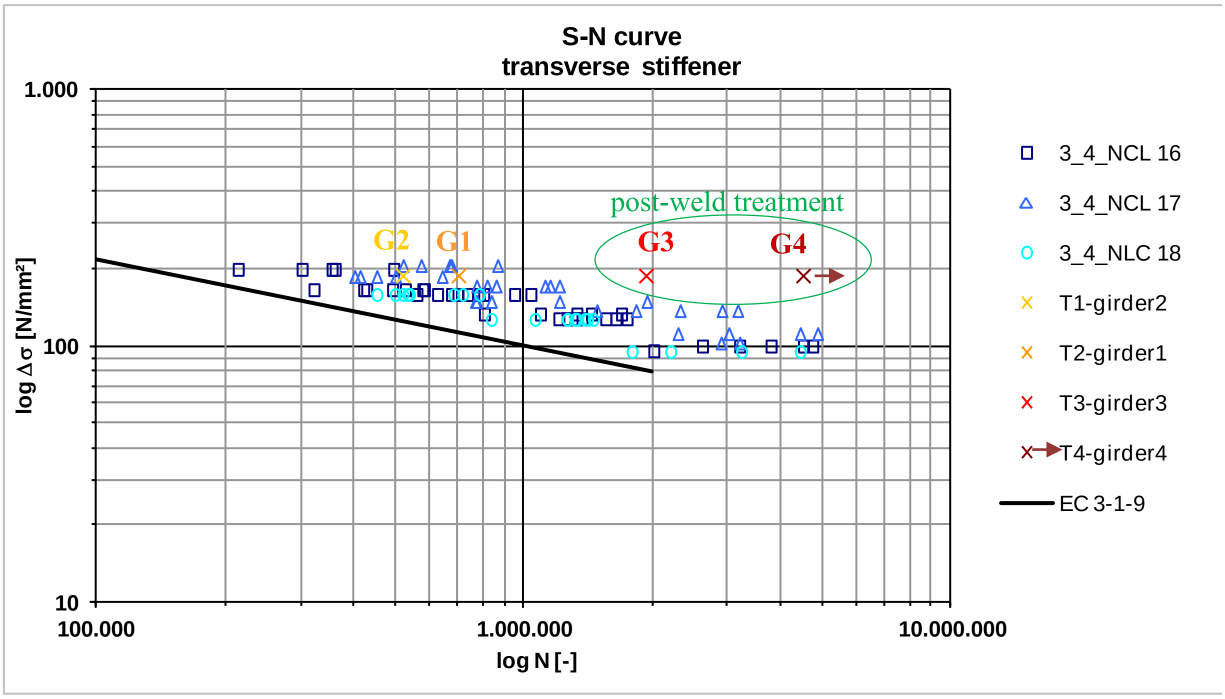


Figure 5-4. Test results compared with S-N curve EN 1993-1-9 (m = 3 and $\Delta\sigma_C$ = 80 N/mm²).

The fatigue tests on the steel girders with transverse stiffeners showed a first cracking of the bearing plate but cracking at the transverse stiffener lead to the failure. Cracks at the bearing plate, being under compression, occur first but the ones being in the tension zone at the stiffener caused fatigue. Failure of the two girders without any post-weld treatment occurred within the range of former tests. Pneumatic impact treatment of the welds at the transverse stiffeners leads to an increasing to more than double of the fatigue resistance

The improvement of the fatigue resistance can be given in relation to the untreated detail by the following equation: $\Delta\sigma_{C.Imp} = \Delta\sigma_C \cdot k$. In S-N-curves this can be realized by a translation (improvement factor k) and / or a change of the slope (m_{Imp}), see Figure 5-5 a) [116].

For the post-weld treatment with high frequent hammering in [116] three concepts are described having in common that the improvement factor depends on the material strength, the stress ratio and the geometry, compare Figure 5-16 b). Following the IIW-Document XIII-22001-07 [96] the slope is kept to m = 3 and the improvement of the fatigue resistance carried out only by a translation of the SN-curve. Different details were analyzed in the REFRESH-project and besides two factors for the stress ratio and the material strength a general factor is introduced. The slope of the SN-curve is changed to $m_{Imp} = 5$. Also Dürr [73] takes this slope into account when introducing the three factors depending on the material k_f, the stress range k_R and the geometry k_L. Table 5-1 shows the factors for the concept following Dürr for the detail of the transverse stiffener.

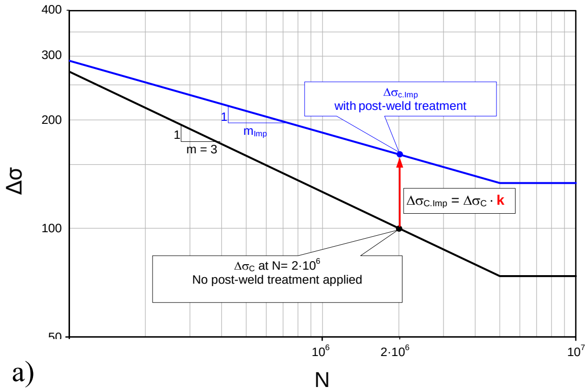


Figure 5-5. Adjustment of S-N curves for improved fatigue resistance [116].

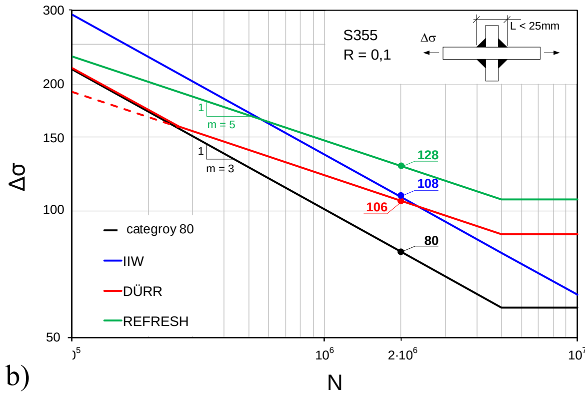


Figure 5-6. Comparison of concepts for the improved fatigue resistance for transverse stiffeners [116].

Table 5-1. Improved fatigue resistance for transverse stiffeners according to Dürr [73]

$\Delta\sigma_{C.Imp} = \Delta\sigma_C \cdot k_f \cdot k_R \cdot k_L$					
material	*kf*	*stress range*	*k_R*	*geometry*	*k_L*
S 355	1,12	R ≤ 0	1,0	L<25 mm	1,25
S 460	1,25	0 < R ≤ 0,5	1 – 4R / 7	25≤L<35 mm	1,12
S 690	1,49	R > 0,5	1,43 (1 - R)	35 ≤ L < 50 mm	1,0
with $m_{Imp} = 5$; $\sigma_{min} \leq 0{,}25\ f_y$ and $\Delta\sigma_{max} = 125$ N/mm²					

The girder tests 3 and 4 with the post-weld treatment by high frequent hammering showed a clear improvement of the fatigue resistance. Keeping the slope at m = 3, as recommended in IIW [96], would lead to higher resistances for low number of cycles, where Dürr and REFRESH are more conservative and m = 5 seems more adequate here.

For the detail of the transverse stiffeners the concept following Dürr [73] was used in SBRI for the design and comparisons of the case studies.

Cracks due to fatigue may result in the loss of structural capacity and may cause costly maintenance actions if they are not detected in time. Inspection intervals are closely linked to the probability of detection of defects and thus influence the sustainability analysis. Non-destructive testing methods (NDT) were applied to the steel girders during the fatigue tests in order to detect cracks and compare the methods regarding application to conditions, such as coating.

By the method of magnetic particle testing (MT) the detection of a crack is possible by introducing a magnetic field and application of particles which follow the streamlines of the field and sediment at the crack. An aperture of the crack to the surface is no precondition of detection. Also under layers of coating a crack should be made visible. The utilization of optimized poles allowing for a closer magnetizing for a 90°-geometry lead to a higher magnetic field. An oily suspension with black particles applied on a steel specimen sprayed with a white contrast colour is a common technique and gives the inspector a facile working. In darkened locations testing under ultraviolet light and fluorescent particles in suspension is a promising way of MT.

The method of penetration testing (PT) uses a coloured ink which infiltrates cracks, after having cleaned the surface in a first step. In a last step a chalky substance is used to make the crack visible at the surface. For penetration testing the major precondition is the crack being open towards the surface. By the amount of colour leaking out of the crack due to the application of chalk an idea of the dimension (depths and/or width) may be given. Here, penetration testing with red and white colours for good visibility were compared to a fluorescent indication colour, made visible under ultraviolet light.

An optimization of detection was performed by applying the innovative solution of ultrasonic testing by Phased Array technique. This ultrasonic testing has no restriction of the testing angle and was compared to MT and PT here. The advantage of no limitation to cracks close to the surface made early crack detection possible. Documentation of the testing is a relevant issue for monitoring a structure during its entire life-cycle. With data achieved by Phased Array such documentation is possible by pictures of the scanned images.

Steel girders in bridges are coated with anti-corrosion coating for protection reasons. The influence of the coating on the crack detection was analyzed. Therefore one side of each girder at mid-span was coated with a standard anti-corrosion coating.

It was shown that by the method of Phased Array (Figure 5-7) it is possible to detect cracks earlier than by the common methods MT and PT and not only at the surface but crack initiation from the weld toe.

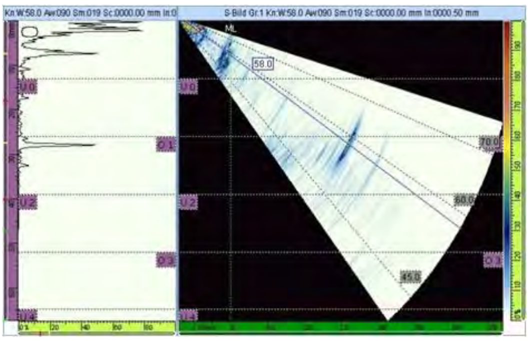


Figure 5-7. Crack detection with Phased Array.

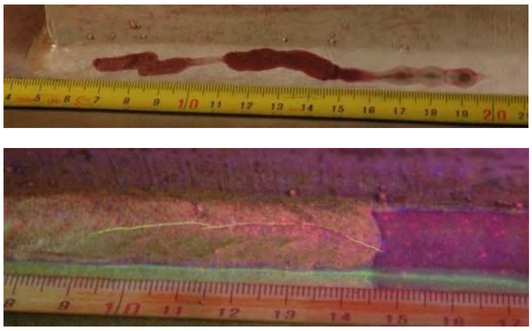


Figure 5-8. Crack detection with PT and MT.

For the penetration testing the crack has to be open towards the surface to enable any penetration. An idea of the crack depth due to the amount of ink leaking out was gained; see Figure 5-8 at the top. Especially the magnetic particle testing under ultraviolet light showed cracks in a very precise and thin way. Its limitation was reached with the coating as no detection was possible there; see Figure 5-8 at the bottom. For further details, compare the test report in [56].

5.2.3 *Fatigue behaviour of horizontally lying shear studs*

5.2.3.1 Description of the detail

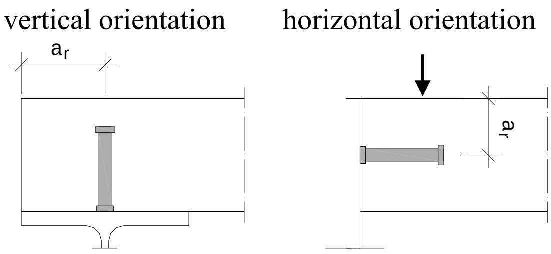


Figure 5-9. Arrangement of headed shear studs with edge influence.

In steel composite bridges shear connections guarantee the interaction between the concrete deck and the steel girder. When dealing with shear studs the aspect of arrangement in regard to the edge distance a_r to the concrete surface needs to be considered, see Figure 5-9. Independent of a vertical or horizontal orientation, studs with a small edge distance a_r may have a reduced resistance due to additional splitting forces [111], [112], [36]. Omitting the upper flange of steel girders and connecting studs directly to the web leads to optimized cross-sections. The behaviour and resistance of the shear studs differ, depending on the load direction and can be divided into longitudinal and transverse shear loading. Mainly substantial fatigue loads derive from concentrated traffic loads caused by wheels passing. This fatigue load causes a transverse loading to horizontally lying shear studs.

Several experimental and numerical investigations [114], [115], [117] on horizontally lying shear studs under static and cyclic loading led to design rules for shear in longitudinal as well as transverse direction, being implemented in EN 1994-2 [78]. A separation regarding the position of the studs (middle or edge position) has to be made. Arranging the studs in a middle position, where horizontal load components are transferred directly, leads to a more favourable condition than in an edge position as shown in Figure 5-9 to the right. An increased resistance with the stud diameter was not observed for horizontally lying shear studs; therefore, in contrast to design rules for headed studs without an edge influence, for horizontally lying shear studs the design is based on forces per stud. A two stage failure mechanism, initiated by concrete crushing close to the weld-collar and concrete splitting, was observed for fatigue loading. For horizontally lying studs a higher concrete damage, forced by a splitting action, leads to lower fatigue strength.

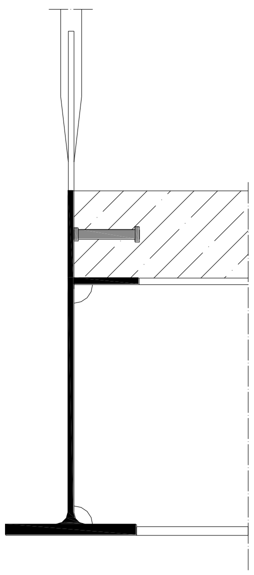


Figure 5-10. Horizontally lying shear studs in a tied-arched bridge.

For steel-composite bridges cracking of the concrete slab and therefore a reduction of the stiffness, is a situation which must be taken into account at design stage. This cracking of the concrete may be caused by negative bending moment e.g. at a support of a continuous composite beam or by tension forces e.g. in a tied-arched bridge due to the axial force in the concrete slab due to the arch action. Where the connection to the main girders is performed by horizontally lying shear studs the critical detail under transverse fatigue loading in cracked concrete due to axial force has to be investigated, see Figure 5-10. The reduction of the concrete strength may lead to reduced fatigue strength because the fatigue failure is strongly influenced by concrete splitting and needs to be considered in design rules. By investigations and own fatigue tests representing the situation of cracked concrete and additional transversal fatigue load, influences are elaborated and a degradation model worked out.

5.2.3.2 Fatigue tests on horizontally lying shear studs

In order to improve existing degradation models fatigue tests on the detail of horizontally lying shear studs in the unfavourable edge position were performed. The influence of a cracked concrete slab was captured by introducing an axial force. Transverse loading, as caused by the wheels of trucks, was applied cyclically and the axial force kept constant. The two load directions were kept independent, where in vertical direction the fatigue load was introduced to steel plates connected by horizontally lying shear studs to the concrete. The cracking of the concrete was achieved by the application of an axial tension force on reinforcement bars. The edge distance of the shear studs was kept constant at a_r = 150 mm throughout the tests, see cross-section in Figure 5-11. The test set-up as well as the applied load directions is shown in Figure 5-12.

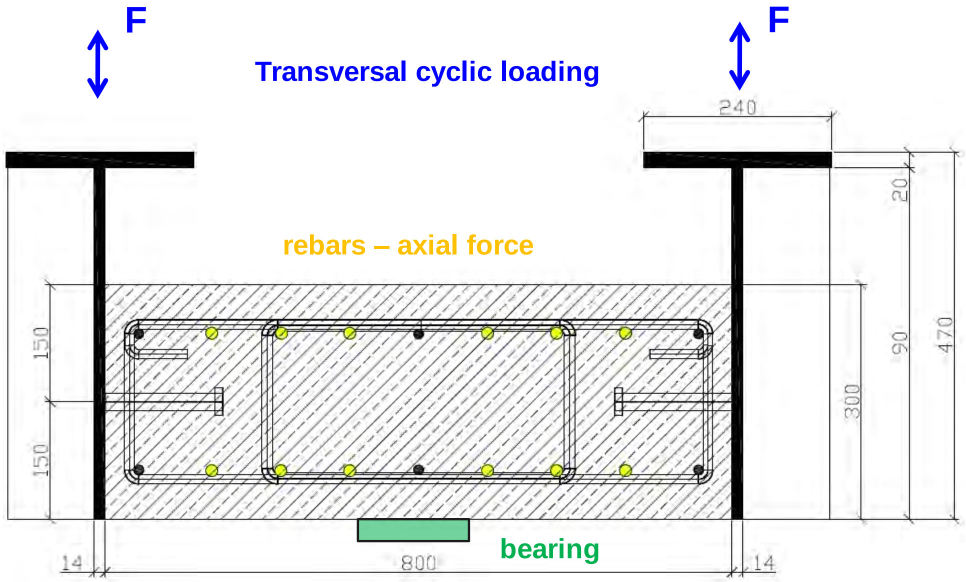


Figure 5-11. Cross-section of the test specimens.

The aim of developing a S-N-curve and therewith describe the fatigue behaviour of the horizontally lying shear studs with the influence of a cracked concrete was achieved by performing seven tests at different stress ranges. A frequency of 3.5 Hz was achieved for the transverse loading while the axial force was kept constant. In 3 static tests the behaviour was studied without axial force and with axial force.

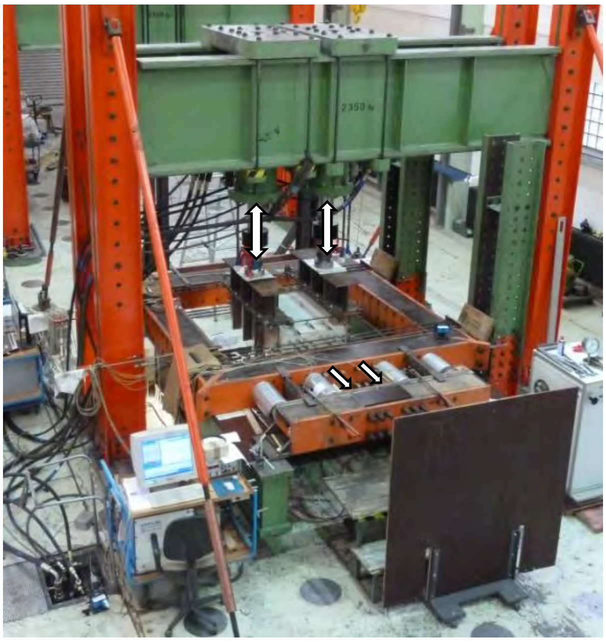


Figure 5-12. Test set-up horizontally lying studs under transversal fatigue load in cracked concrete.

The failure of the test specimens was defined either as a steel shearing of the studs or a concrete break-out, see Figure 5-13. Besides one of the tests, the concrete break-out due to the additional splitting force at the reduced edge distance initiated the fatigue failure of the test specimens.

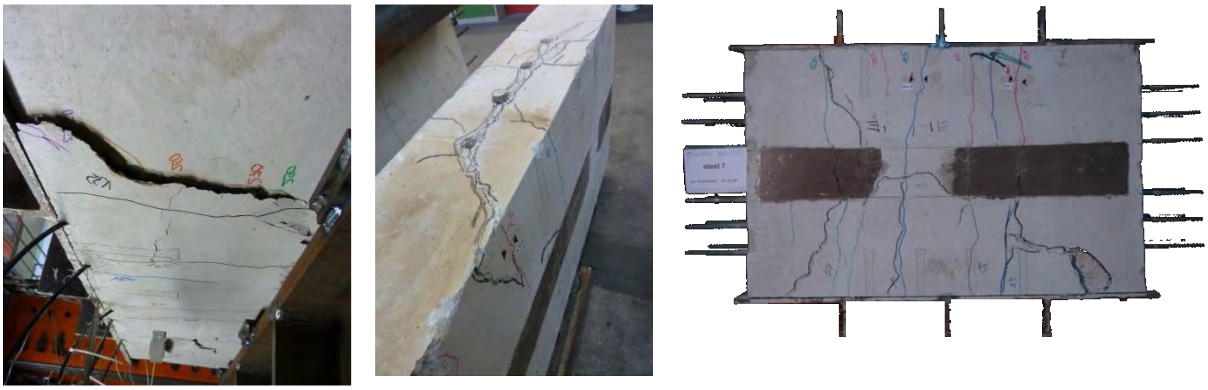


Figure 5-13. Concrete cracking at failure; on the shear surface and cracking pattern of the specimen.

The fatigue failure of the here performed tests with additional axial force causing cracked concrete needs to be compared to former tests [114], [115], [113] without cracked concrete. In Figure 5-14 it is made visible that the influence of the cracked concrete leads to a reduction of the fatigue resistance, see lower line of the SN-curves. It can be concluded that at design stage it should be taken into account that with an expected cracking of the concrete a reduced fatigue resistance of horizontally lying shear studs is given, [27]. For further details compare [55].

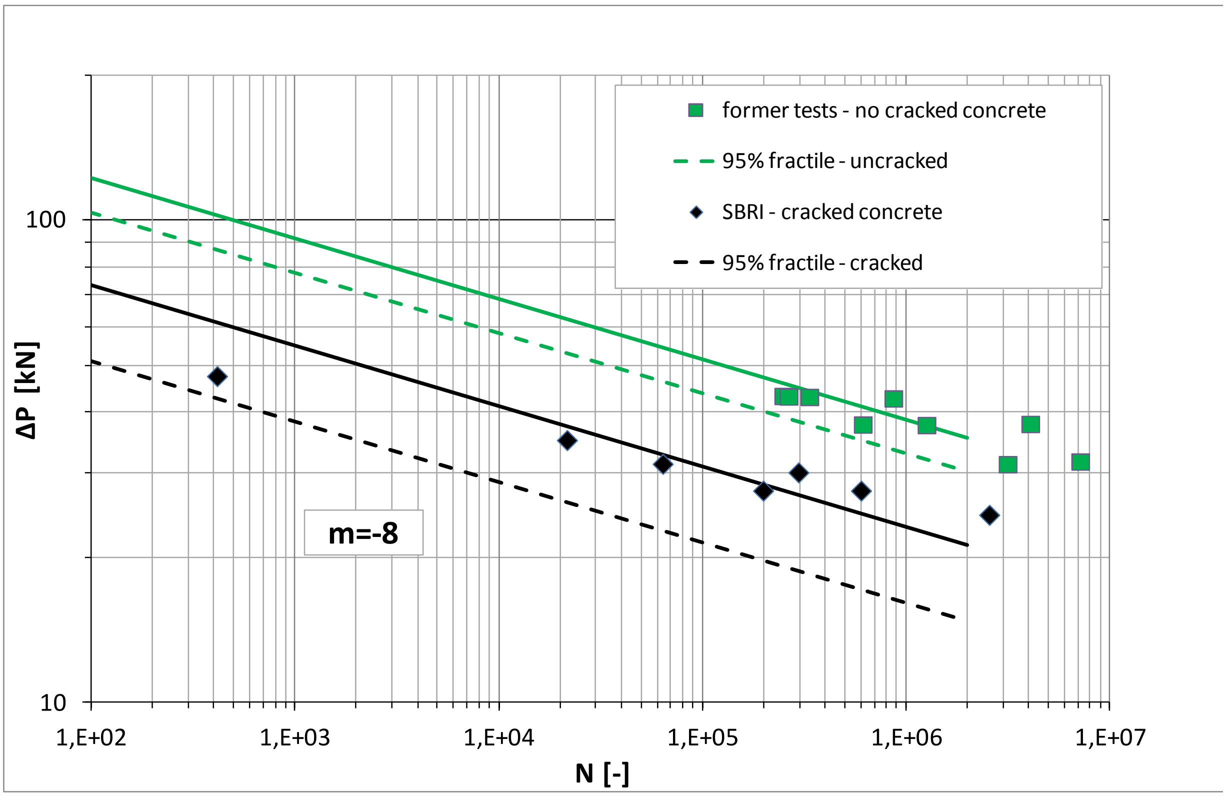


Figure 5-14. Comparison of test results with and without cracked concrete.

5.2.4 *Conclusions from fatigue tests*

The fatigue tests on girders with transverse stiffeners showed that an increase of the resistance can be reached by the application of high frequent hammering as post-weld treatment. With this method the life-cycle performance of this typical detail can be improved and the lifetime increased. A comparison of non-destructive inspection methods showed on the one hand limitations of methods when applied to coated steel surfaces and on the other hand the advantages of the innovative method of phased array technique with early crack detection.

The investigations on the fatigue behaviour of composite sections with horizontally lying shear studs demonstrate the significant influence of cracked concrete. The reduced fatigue resistance is to be taken into account for the life-cycle design of this detail.

Detailed information on the girder tests as well as on the tests on horizontally lying shear studs is given in the test reports [56], [55].

Final interpretations were included in other work packages in form of improved fatigue rules for PWT, recommendations for the application of non-destructive testing methods, SN-curves and resistance checks for horizontally lying studs.

5.3 Corrosion

5.3.1 *Description of the problem*

A common problem found in composite bridges is corrosion attacks in the joint between the steel beam and the concrete deck as illustrated in Figure 5-15.

Rust stains may occur in the joint between the steel beam and the concrete deck and/or on the surface of the sealant in the areas where sealant is used in the interface zone. According to [33], rust stains are caused by corrosion on the steel surface behind the sealant or by corroding binding wire. In the latter case, the stains have no direct influence on the condition of the steel beam. Once corrosion is noticed, a proper repair is needed in order to stop the process and avoid future major problems.

Figure 5-15. Corrosion in the interface steel-concrete [33].

To repair the corrosion problem in the joint between the steel beam and the concrete deck different methods can be used, such as the application of surface coatings or of elastic sealants to the joint.

5.3.2 *Corrosion test preparation*

To assess the best strategy to overcome this problem, cyclic corrosion tests were carried out at the Laboratory of Electroanalysis and Corrosion of *Instituto Pedro Nunes* (University of Coimbra). These tests were performed in a Cyclic Corrosion Test Chamber, as illustrated in Figure 5-16.

Figure 5-16. Corrosion Test Chamber ASCOTT CC450xp.

The Cyclic Corrosion Test Chamber ASCOTT CC450xp allows the realization of accelerated cyclic corrosion tests, with or without the presence of sulphur dioxide, besides the traditional cyclic corrosion tests (duration from several hours upwards – no upper limit).

Due to limitations in the dimensions of the chamber and in the maximum weight of the specimens, a prototype made of concrete and steel was used (weight = 2.85 kg).

The steel plates are illustrated in Figure 5-17 before the concrete plus coating system is applied.

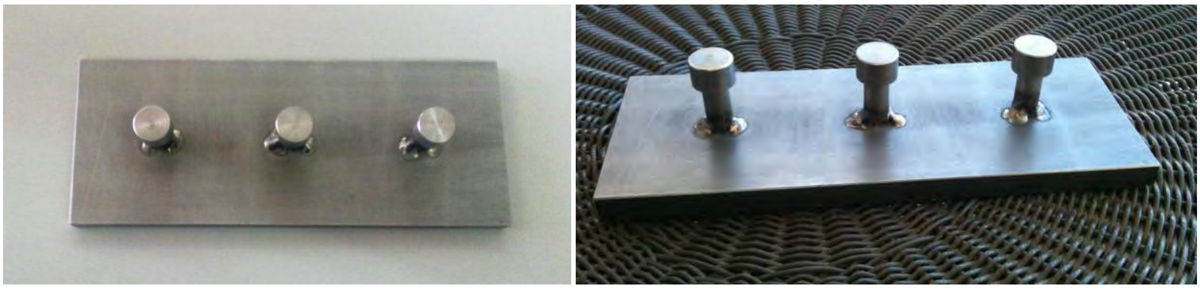


Figure 5-17: Steel plates before the application of the coating system.

The tests were performed using different types of coating systems, as indicated in Table 5-2.

Table 5-2. Coating system in each test.

Type of coating	Coating system	Comment
Type 1	Hempadur Zinc 15360 @ 50 µm	The interface between steel and concrete is not coated
Type 2	Hempadur Zinc 15360 @ 50 µm	
Type 3	Hempadur Zinc 15360 @ 50 µm Hempadur 15570 @ 100 µm	The interface between steel and concrete is coated only with the first coating
Type 4	Hempadur Zinc 15360 @ 50 µm Hempadur 15570 @ 100 µm	
Type 5	Hempadur Zinc 15360 @ 50 µm Hempadur 15570 @ 100 µm Hempathane 55210 @ 50 µm	

Four tests were performed, using the five specimen types each time, with 1% NaCl salt spray and involved 8 h cycles consisting of:

Test 1

Step 1: 18 cycles each of 8 h:

- salt spray: 30 min exposure in saline environment at 35°C and 1.0% NaCl pH 6.5.
- moisture and controlled drying: the samples are exposed to a temperature of 35ºC with 95% RH during 1 h 30 min and then to a temperature of 35ºC with 55% RH during 2 h 30 min.
- drying: 3 h 30 min exposure at 35º C, 20% RH.

Step 2: 3 cycles of 8 h each:

- salt spray: 2 h exposure in saline environment at 35°C and 1.0% NaCl pH 6.5.
- drying: samples are exposed to a temperature of 60ºC with 20% RH during 4 h.
- moisture: 2 h exposure at 50º C with 95% RH.

Test 2: 63 cycles of 8 h, each comprising:

- salt spray: 2 h exposure in saline environment at 35°C and 1.0% NaCl pH 6.5.
- moisture: samples are exposed to a temperature of 50ºC with 95% RH during 2 h.
- drying: 4 h exposure at 60º C with 20% RH.

Test 3a: 42 cycles of 8 h:

- each cycle as in Test 2

Test 3b: 63 cycles of 8 h:

- each cycle as in Test 2

The number of cycles undertaken depends on the condition of the corroded specimens. According to the results obtained in Test 1, the conditions for the subsequent cyclic corrosion tests were adapted. Visual and microscopic examination of specimens before and after corrosion was carried out.

Further details may be found in the internal report on the corrosion tests, [14].

5.3.3 *Test results: analysis of corrosion data from test 3b (63 cycles)*

A more detailed analysis was done of the samples in Test 3b after 63 cycles of corrosion, in which the samples were carefully prepared with no cement remaining on the coated surface. Table 5-3 concerns the appearance of iron oxides i.e. red colouration (interface refers to the coated steel/concrete interface, edges refer to the upper corners of the coated steel, surface refers to the upper steel surface).

Table 5-3. Visual semi-quantified analysis of corrosion.

Sample Type	Interfaces	Edges	Surface
1	Side 1: 100 % Side 2: 20 % Ends: 20 %	Side 1: 2 % Side 2: 5 %	6 points 2 mm diameter Whole surface whitened
2	Very slight discoloration on one side	Side 1: 4 lengths of 2 mm Side 2: 1 5mm and 1 12 mm	2 points diameter 0.5 cm 6 points 2 mm Whitened surface
3	---	One corner: total One end: 2mm length	Very slight discoloration
4	Very slight discoloration on one side	One corner: 2 mm	---
5	Very slight discoloration on one side	---	---

There is a clear improvement from Sample Type 1 up to Sample Type 5. However, apart from the whitening of the surface of the coatings for Samples 1 and 2, all other corrosion is localised (which would make treatment easier).

These data can be analysed in the light of Norm ISO 12944-2 [102] which classifies different atmospheres that can cause corrosion, specifically corrosion protection of steel structures by protective paint systems. The classification follows Table 5-4 and is done with respect to mass loss.

Table 5-4. Categories of environment (ISO 12944-2) [102].

Corrosivity category		Environment (guiding examples)
C1	very low	Indoor spaces with occasional condensation Outdoor air: inland rural
C2	low	Dry indoor spaces
C3	medium	Indoor spaces with high moisture content, not much impurities Outdoor air: inland urban, mildly saline
C4	high	Indoor spaces: chemical industry, swimming pools, seaside docks Outdoor air: inland industrial plants, seaside urban areas
C5-I	very high	Outdoor air: very humid industrial atmosphere
C5-M	very high	Outdoor air: saline seaside atmosphere

The tests undertaken and the degree of corrosion exhibited do not permit any deductions of mass loss. However, in the worst cases the atmospheric conditions may correspond to C5-M.

According to the usual norms, the durability is expressed as low (2 - 5 years), medium (5 - 15 years) and high (> 15 years).

The durability can be estimated as follows:

A. In a C5-M environment
   - Sample Type 1: low (2 - 5 years)
   - Sample Type 2: low (2 - 5 years)
   - Sample Type 3: medium (5 - 15 years)
   - Sample Type 4: high (>15 years)
   - Sample Type 5: high (>15 years)

B. In a C3 environment:
   - Sample Type 1: medium (5 - 15 years)
   - Sample Type 2: medium (5 - 15 years)
   - Sample Type 3: high (>15 years)
   - Sample Type 4: very high (> 20 years)
   - Sample Type 5: high (> 20 years)

It should be noted:

A. Comparing the extent of corrosion over Test 3b, it is clear that the major part appears near the beginning of the test (9 cycles) and does not progress, suggesting that the zones of corrosion do not increase by corrosion under the coating (the adhesion remains excellent in all cases) and that there is some protection from the oxide layer that has been formed. Nevertheless, in an outdoor situation, it may be dangerous to assume that this will be helpful since the atmosphere may be acidic and thus tend to dissolve any metal oxides.

B. If there are defects in the preparation of the composite structures, corrosion will be greater. It was noticed in some of the samples, that concrete leaks onto the coating surface in the mould during preparation, and the coating is partially destroyed on removal of the excess concrete from the surface leading to extensive corrosion at those points. Such situations must be avoided.

The data analysis above is only valid if there are no preparation defects in the coatings.

5.3.4 *Conclusions from corrosion analysis*

As expected, the most complete coating system led to the best performance. However, additional information is necessary in order to predict the service life of different coating strategies.

The coating system used in the tests described in the previous paragraphs corresponded to that used in one of the case studies performed in this project. In fact, the coating system defined for case study B2 was the coating system of the Type 5 sample, see Table 5-2. According to the results of the tests, there is no major difference between the Type 4 and Type 5 coating system, which corresponds to standard coating systems of motorway bridges. Nevertheless, preparation defects in the coatings must be rigorously avoided for this to be valid. Considering an environmental category with medium corrosivity (C3), see Table 5-4, the service life of the coating system Types 4 and 5 is estimated to be more than 20 years, which provides useful information for the prediction of the maintenance plan of composite bridges.

5.4 Carbonation

5.4.1 *Introduction*

Long-term durability of reinforced concrete (RC) structures has become one major concern in view of the vast amounts of money required to maintain the infrastructures in a serviceable state. Regarding the steel RC corrosion, resulting from chloride ingress and/or atmospheric carbonation, the traditional approach for the concrete design has been to follow deemed-to-satisfy rules which set requirements on mix-parameters, thickness of the concrete cover, crack width limitations, etc. However, these requirements are no longer appropriate because of the complexity and the variety of the binders used today, and even stifle the designers who have nowadays numerous possibilities in terms of mix-design parameters (use of admixtures like superplasticizers, air-entraining agents, etc., and use of cement blended with supplementary cementitious materials like fly ash, slag, silica fume, etc.). That is why a need is currently appearing for performance-based approaches [60], [58], in which the rules are associated with the performance to be achieved in terms of durability properties (*i.e.*, porosity, permeability, etc.).

Corrosion of the embedded reinforcement steel, resulting from atmospheric carbonation, is a matter of considerable concern which irreversibly affects the serviceability of RC structures. Most concrete structures are exposed to the action of CO2 which diffuses into the concrete cover, dissolves in the pore water, and reacts with the hydration compounds, causing a reduction in the pH-value which thus makes corrosion of the steel reinforcement possible [132]. This issue is particularly pronounced for cementitious materials with a low portlandite content (CH) since CH is the main supplier of alkaline buffering capacity. Therefore an ordinary concrete (medium to high porosity) made of a binder with a large amount of supplementary cementitious materials is likely to be more sensitive to carbonation. This is why the quantification of the carbonation mechanism for these kinds of concrete is crucial, all the more since their use will drastically increase in the next decades to fulfill commitments related to the mitigation of the CO2 footprint.

The simplest and most effective way of enhancing service life (SL) of RC structures is to increase the length of the corrosion initiation period which is defined as the time required for the first layer of steel rebars to become depassivated. To make the prediction of this induction period possible, mathematical models can be used. Most of the time, a deterministic approach is adopted. Even if a deterministic approach can provide an acceptable assessment of the carbonation penetration for accelerated conditions, the predictions for durations of more than fifty years are very uncertain given that most input data of the model show a great variability which rejects any idea of absolute reliability.

A carbonation model [59] is studied in the framework of a time-dependent reliability approach. The extended version of the Bakker model is based on the same description of carbonation as the one used in the Papadakis model [127], but it is moreover able to take into account the influence of wetting-drying cycles which can slow down the rate of carbonation [133]. The main input data required for this model are the durability indicators (DI) φ (accessible-to-water-porosity) and Kl (intrinsic liquid-water permeability), the moisture parameters λ, μ (associated with desorption properties) and RHext (external relative humidity).

5.4.2 *The Bakker model*

The Bakker's carbonation model takes into account the influence of drying-wetting cycles on the carbonation mechanism of RC structures [59]. It is considered that carbonation is stopped when the relative humidity (*RH*) prevailing in the pores is higher than a threshold value RH_{lim} = 80%. Figure 5-18 illustrates the modelling steps of the carbonation penetration.

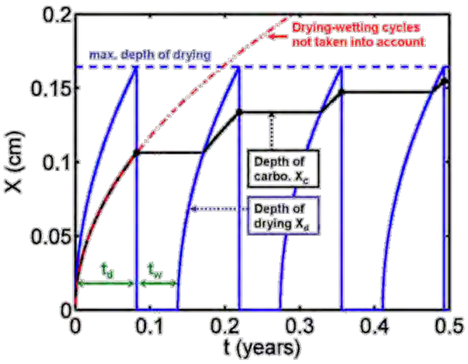


Figure 5-18. Carbonation depth profile according to the Papadakis' and Bakker's model.

During the drying step (duration t_d), carbonation classically evolves according to equation (5-1). Then, concrete wets instantaneously and stays saturated during a period t_w stopping the carbonation mechanism. During the next drying step, carbonation starts again when $RH > RH_{lim}$ in the vicinity of the carbonation front. During the drying periods (thus including carbonation), the external relative humidity is set and equal to RH_{ext}. RH_{ext} defines a liquid-water saturation S_{ext} according to the desorption relation, see equation (5-2). To quantify the carbonation penetration, the liquid water saturation is considered constant in equation (5-1) and equal to S_{ext}. During the drying steps, the determination of the time evolution of the depth X_d at which $RH = 80\%$ is essential to quantify the carbonation penetration. $X_d(t)$ is assessed by using a model of moisture transfers which considers that the liquid-water movement due to capillary gradients only contributes to the moisture transfers occuring during the drying step. It is reminded that this assumption is valid in the case of weakly permeable porous media like cementitious materials [123] and leads to a diffusion-like equation written as follows for a 1-D configuration:

$$\frac{\delta S}{\delta t} = \frac{\delta}{\delta x} D(S) \frac{\delta S}{\delta x} \qquad (5\text{-}1)$$

Where

$$D = -\frac{K_l}{\varphi} \frac{\rho_l RT}{\mu_l M_w \mu_l} \frac{1}{RH} \frac{dRH}{dS} k \qquad (5\text{-}2)$$

Where ρ_l is the density of the liquid-water, R the ideal gas constant, T the absolute temperature, μ_l the dynamic viscosity of liquid-water, M_w is the molar mass of the water molecules, K_l is the intrinsic permeability to liquid-water of the studied concrete, and $k(S)$ stands for the relative permeability to liquid-water determined by means of the Mualem's theory [125]:

$$k(S) = S^{1/2}\left[1-\left(1-S^{1/\mu}\right)^{\mu}\right]^{2} \qquad (5\text{-}3)$$

The boundary and initial conditions correspond to a 1-D configuration: i.e., $S(x = 0, t > 0) = S_{ext}$ and $S(x > 0, t = 0) = 1$. The previous differential equation (5-3) can be simplified in the case of a semi-infinite medium (i.e., if it is assumed that the core of the concrete will never be affected by the moisture transfers and hence stays always saturated). Thus, by letting $y = x/(2\sqrt{t})$, it is possible to exhibit a self-similar ordinary differential equation (5-4) governing each drying step:

$$2yS'(y) + D'(S)S'(y)^2 + D(S)S''(y) \tag{5-4}$$

where $S(y = 0) = S_{ext}$ and $S(y \rightarrow +\infty) = 1$. As a result, the drying depth X_d theoretically follows a square root of time law: $X_d = d\sqrt{t}$ where d is numerically determined by solving this ordinary differential equation with a finite difference scheme. The coeffient d depends on the following variables: porosity φ, liquid-water permeability K_l, van Genuchten's parameters λ and , RH_{ext} and RH_{lim}.

5.4.3 *Probabilistic approach*

The Bakker model can easily be developed in a probabilistic framework by calculating a time-dependent probability P_f of corrosion initiated by carbonation. P_f is defined as the probability that the carbonation front exceeds the concrete cover E by taking into account the variability of the input parameters. In other words, $P_f(t) = P(M_t = E - X_C(t) < 0)$ where M_t refers to the safety margin at time t. The reliability index β has been determined by using the Rackwitz-Fiessler algorithm. According to the first order approximation (FORM), P_f is calculated by means of the relation $P_f = \Phi(-\beta)$ where Φ is the cumulative function of the standard normal distribution.

The reliability calculations are illustrated through two concretes C1 and C2 designed for a SL of 100 years. C1 and C2 are of class C34/45. The environmental exposure class, in accordance with the European standard EN 206 [79], is assumed to be XC4, i.e., the most severe sub-class for a corrosion risk induced by carbonation (concrete surface subject to wet and dry periods). According to the Eurocode 2 [76], and its French Annex in the case of a concrete bridge deck (SL>100 years), the recommended structural class is S5 for the two studied concretes. As a consequence the concrete cover is fixed at E =3.5 cm for the two studied concretes. The main characteristics of these concretes are provided in Table 5-5. The used binder is an OPC cement CEM I 52.5 PM ES. Concrete C2 is prepared with fly ash.

Table 5-5. Main characteristics of the studied concretes: cement content C and fly ash content FA. 28-day compressive strength Rc28. Accessible-to-water porosity φ, CH content n0, intrinsic liquid-water permeability Kl, and desorption properties λ and μ. The DI are determined after a 90-day water curing period.

Concretes	C1	C2	COV [%]
C [kg.m−3]	300	223	
FA [kg.m−3]	95	80	
w/b [-]	0.62	0.52	
Rc28 [MPa]	41	48	
φ [%]	14.4	13.9	6.5%
-log(Kl) [m2]	19.5	19.9	3%
n0 [mol.L−1]	1.25	0.91	10%
λ [-] & μ [-]	9.5/0.45	8.2/0.44	10%

To perform reliability calculations to assess the service life (relevant for the design of new structures) or the residual service life (relevant for the re-design of existing structures), a target reliability level shall be adopted, in dependence of the consequences of the failure (i.e., when carbonation reaches the first layer of rebars) in terms of risk to life or injury (very low here), the potential economic losses (moderate), the degree of societal inconvenience (moderate), and the amount of expense and effort required to reduce the risk of failure (high). That is why, the target β-value, related to the corrosion risk due to carbonation, reasonably refers to a serviceability limit state. According to Eurocodes, a minimum reliability index β_{lim} of 1.0 should be considered for a service life of 100 years. Figure 5-19 illustrates the profiles of the reliability indices for the two studied concretes. For $\beta_{lim} = 1.0$, the predicted service life is around 65 years for C1 and over 100 years for C2. With this model, only the concrete C1 requires specific actions to reach the service life of 100 years.

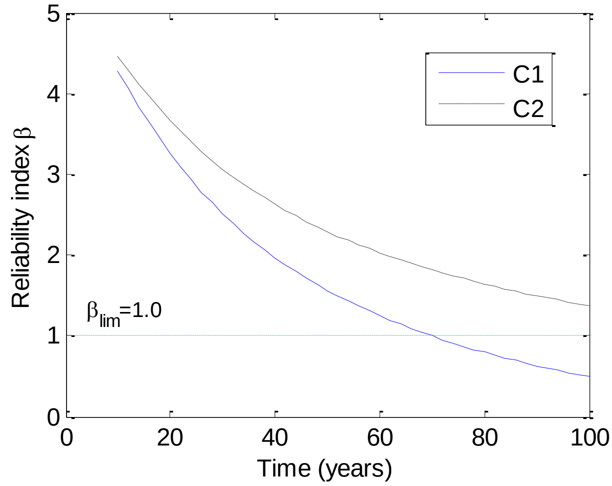


Figure 5-19. Reliability index profiles for C1 and C2.

5.4.4 *Determination of optimal inspection/maintenance times*

This study proposes a probabilistic framework for optimizing the timing of inspection and maintenance regarding the carbonation-induced corrosion risk. The reinforced concrete deck of case study A, analyzed here in the SBRI project, is considered in this section as illustration. The inspection technique corresponds to the measurement of the carbonation depth by spraying phenolphthalein on concrete cores which are freshly split after drilling in the deck (in eight randomly spaced points). At each inspection time ti, the decision to establish a repair procedure is determined by assessing the event margin $H_{ti} = \gamma \times E - X_C(ti)$. The γ coefficient represents a detection threshold and implies that the structure is repaired before corrosion initiation has been reached (in the present research $\gamma = 0.9$). If the inspection indicates that the repair criterion is reached, the repair consists in rebuilding the carbonated concrete cover by a repair mortar. Three different repair mortars have been selected (see Table 5-6) with three different qualities.

Table 5-6. Main characteristics of the studied mortars (see Table 5-5 for notations).

Mortars	M1	M2	M3	COV [%]
C [kg.m−3]	492	757	528	
FA [kg.m−3]	-	-	160	
w/b [-]	0.38	0.34	0.67	
Rc28 [MPa]	-	++	+	
Sustainability	+	-	++	
φ [%]	20.1	18.3	16.1	6.5%
-log(Kl) [m2]	18.5	18.8	19.5	3%
n0 [mol.L−1]	1.85	2.31	1.05	10%
λ [-] & μ [-]	12.1/0.44	8.1/0.48	3.8/0.5	10%

The effect of the repair is to return the cover thickness to its initial value, i.e., E =3.5 cm for the two studied concrete structures made of C1 and C2. Nevertheless an increase of the variability E is taken into account to reflect the lower workmanship quality of achieving a fixed cover thickness with a mortar (COV= 15% instead of 13%). A time t, the same safety margin $M_t = E - X_C(t)$ is used to assess the failure probability. Such a maintenance policy gives rise to a binary event tree. For t<t1, t1<t<t2 and t2<t<t3, the failure probabilities are provided by equations (5-5), (5-6) and (5-7), respectively.

$$P_f(t) = P(M_t < 0) \tag{5-5}$$

$$P_f(t) = P(H_{t1} < 0 \cap \hat{M}_{t-t1} < 0) + P(H_{t1} \geq 0 \cap M_t < 0) \tag{5-6}$$

$$\begin{aligned} P_f(t) = & P(H_{t1} < 0 \cap \hat{H}_{t2-t1} < 0 \cap \hat{M}_{t-t2} < 0) \\ & + P(H_{t1} < 0 \cap \hat{H}_{t2-t1} \geq 0 \cap \hat{M}_{t-t1} < 0) \\ & + P(H_{t1} \geq 0 \cap \hat{H}_{t2} < 0 \cap \hat{M}_{t-t2} < 0) \\ & + P(H_{t1} \geq 0 \cap \hat{H}_{t2} \geq 0 \cap M_t < 0) \end{aligned} \tag{5-7}$$

Where $\hat{M}$ or $\hat{H}$ stand for the safety and event margins when carbonation occurs through the repair mortar whose thickness is characterized by a higher COV (15%) than the initial concrete cover.

Figure 5-20 illustrates the reliability index profile β for C1 and C2 concretes in the case of a double inspection at t1 =70 years and t2 =90 years. It appears that the earlier the inspection (70 years vs. 90 years), the less obvious the rise of β. Moreover, the observation is readily drawn that beyond the inspection/repair time the decrease of β is more pronounced because the quality of the repair mortar regarding the carbonation ingress is lower than the quality of the initial concrete cover. This is particularly noticeable in the case of M1 which is of lower quality than M2 and M3. The minimum value of β which is reached during the whole service life heavily depends on the time of initial inspection and the kind of mortar which is used for the repair. As a result, the minimum β-value βmin can be achieved at the end of the targeted service life (100 years) or before, as illustrated in Figure 5-20.

Because targeted inspections are time consuming and expensive, they should be planned and timed for when they produce the most benefit. In the present research, the inspection times are determined by minimizing the total costs of inspection and repair (CI+CR), and by maximizing the reliability index at 100 years. The costs of inspection are

$$CI = \sum_{i=1}^{N} \frac{C_i^0}{(1+r)^{ti}} \tag{5-8}$$

where C_i^0 are the costs of an inspection at time t = 0 and r is the discount rate (2%).

The costs of repair are equal to

$$CR = \sum_{i=1}^{N} \frac{p_i C_R^0}{(1+r)^{ti}} \tag{5-9}$$

where C_R^0 are the initial costs of repair (for mortar M1 435 k€, M2 670 k€ and M3 467 k€). p_i is the probability of detection of repair at time ti: $p_1 = P(H_{t1} < 0)$ and $p_i = P(\hat{H}_{ti+1-ti} < 0)$ for i ≥ 2.

Figure 5-21 shows different solutions of maintenance of the concrete structure C1 according to the inspection timing and the mortar quality (M1, M2 or M3). Different inspection intervals have been tested from 10 to 50 years, as well as different moments of first inspection (20, 30,…, 80 or 90 years). The coordinates for each point correspond to the minimum β-value and the costs of maintenance (CI+CR). The principle of the procedure is to identify the point with the lowest costs able to ensure βmin >1.0. It comes that the optimal solution is provided by a double inspection at years 60 and 80 and a maintenance using M3 as the repair mortar.

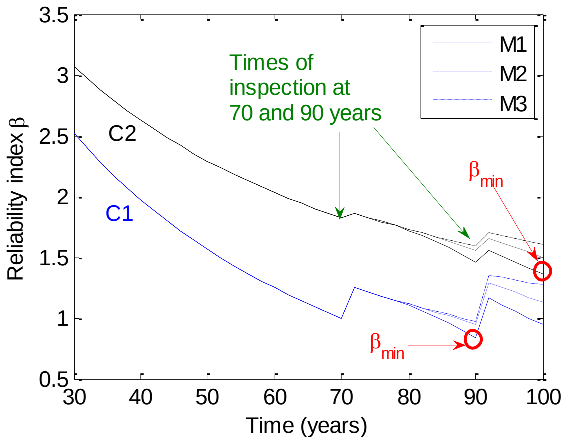


Figure 5-20. Influence of two inspections at 70 and 90 years on the time-dependent reliability index β for concretes C1 and C2.

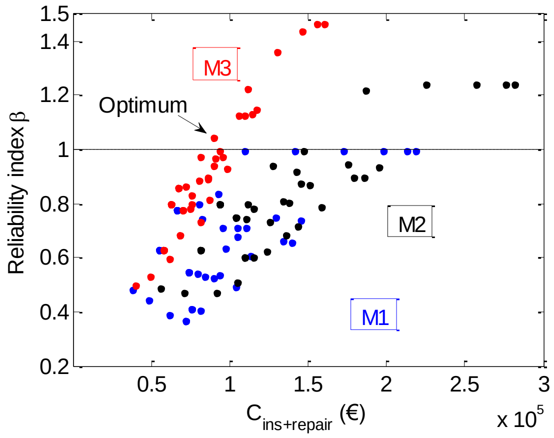


Figure 5-21. Optimization of the timing inspections according to the mortar quality for concrete C1.

5.4.5 *Conclusions from carbonation analysis*

The development of the Bakker model in a probabilistic framework provides a powerful tool to guide for the choice of the maintenance timing and the quality of the repair mortar when the optimization function is defined in terms of costs (inspection and repair) and safety. It is noted that this procedure can still be improved by allowing the possibility of changing the repair mortar quality between the different inspection times.

Further details may be found in the internal report on the carbonation analysis, [20].

6 CASE STUDIES – INTEGRAL APPROACH

6.1 Case A – medium span bridges

6.1.1 *Life-cycle environmental analysis*

6.1.1.1 Introduction

Life-cycle environmental analysis was carried out according to the methodology described in section 4.2.

The analysis takes into account all the stages, from material production to end-of-life stages, as illustrated in Figure 4-3. However, no traffic interference was considered for the construction and end-of-life stages. In the former case, it was assumed that being a new bridge, there was no traffic on the bridge. In the latter, it was considered that traffic would be diverted to an alternative route. In addition, due to the lack of data, the use of construction equipment was not considered in the analysis.

In the operation stage, for the quantification of environmental impacts due to traffic congestion, two alternative scenarios were considered: (i) a "day work" scenario, in which maintenance actions take place during the day (8 a.m. to 5 p.m.); and (ii) a "night work" scenario, in which maintenance actions take place during the night (9 p.m. to 6 a.m.).

In this report, only the results of the analysis considering the "day work" scenarios are provided. The detailed results of the analysis are provided in the internal report, [17].

6.1.1.2 Reference case study A0

The results of the life-cycle analysis of the reference case study A0 are summarized in Table 6-1, taking into account the "day work" scenario. For a better understanding of the contribution of each stage to the aggregated result, these results are also represented in the contribution graph of Figure 6-1.

Table 6-1. Life-cycle results per life-cycle stage ("day work" scenario).

Impact category	Unit	Total	Materials	Construction	Operation	End-of-life
ADP	kg Sb eq	65 691.50	23 364.45	113.09	46 916.44	-4 702.48
AP	kg SO_2 eq	23 349.96	11 726.09	47.41	13 141.09	-1 564.63
EP	kg PO_4^{3-} eq	2 682.89	1 139.61	8.46	1 480.66	54.16
GWP	kg CO_2 eq	5 160 431.22	3 921 644.10	15 727.81	2 007 968.80	-784 909.49
ODP	kg CFC-11 eq	1.11	0.17	0.00	0.93	0.01
HTP	kg 1,4-DB eq	1 049 398.03	421 936.91	2 255.93	599 335.45	25 869.74
TETP	kg 1,4-DB eq	1 147.39	556.18	2.99	541.60	46.61
POCP	kg C_2H_4	3 872.52	1 268.02	1.96	3 081.50	-478.97

The material production stage is the stage that most contributes to the impact category of global warming (GWP) with a percentage above 50%. On the other side, this stage has a minimum contribution to impact category ozone depletion (OD), with a percentage of about 15%. The operation stage has a major contribution to most impact categories except GWP. It is noted that for the operation stage the processes included are the production of new materials (when applicable) and the disposal of waste materials.

Stage of construction has a negligible contribution for all impact categories; while end-of-life stage has a global contribution of less than 10%.

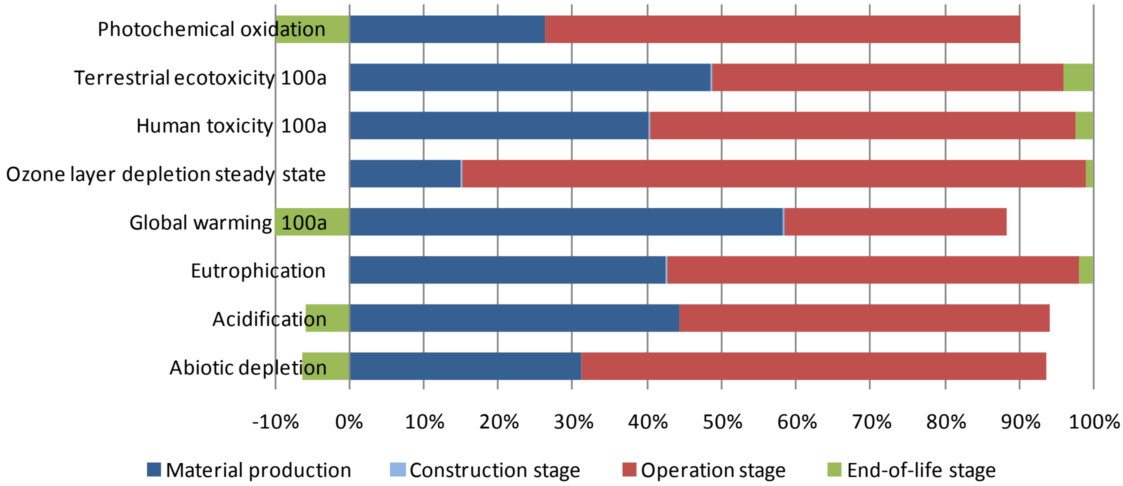


Figure 6-1. Contribution of each stage to impact category ("day work" scenario).

The results presented in Figure 6-1 were rearranged according to the main process involved in the life-cycle analysis and the results are represented in Figure 6-2. Four main processes were identified: (i) production of material, (ii) transportation of materials, (iii) disposal of materials, and (iv) traffic congestion. According to Figure 6-2, the processes of production of materials and traffic congestion are dominant for most impact categories. The production of materials has a contribution above 50% for most impact categories.

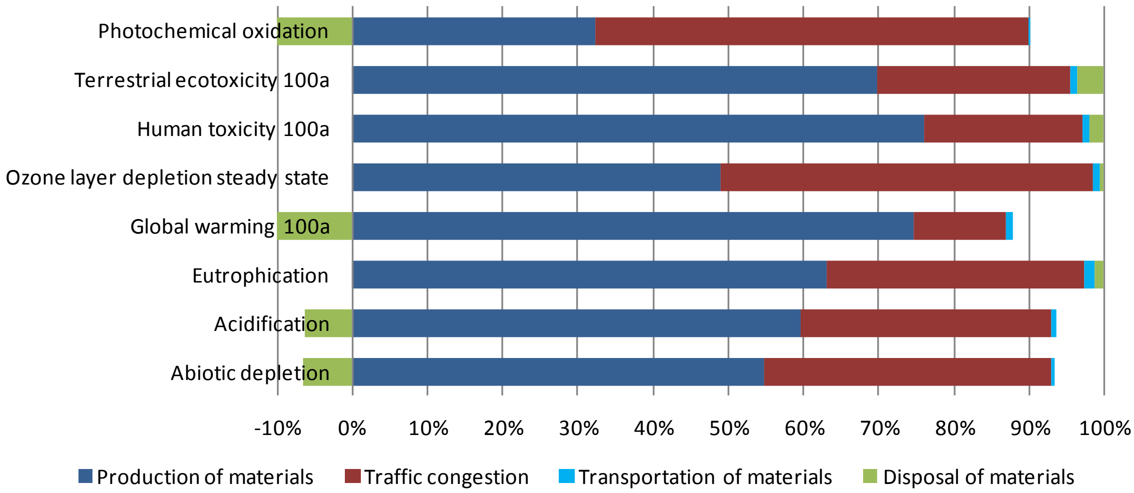


Figure 6-2. Contribution of each main process to impact category ("day work" scenario).

6.1.1.3 Aggregate life-cycle results for variants A1, A2 and A3

The results obtained for the variant case studies A1, A2 and A3 are represented in Table 6-2, considering the "day work" scenario in all cases. This table also indicates the variation of the results for each case study in relation to the reference case study A0.

Case study A1 has a small advantage in relation to case study A0 due to the fact that the use of HSS enables to reduce the amount of structural steel and thus reducing the environmental impacts due to the construction stage. For the same reason, case study A2 has a small disadvantage in relation to case study A0 as the use of self-weathering steel implies a small increase in the amount of structural steel. In should be noted that, in this particular case, the use of self-weathering steel has no major advantage because no traffic is considered under the bridge and thus the reduction in maintenance needs has no effects on traffic.

Table 6-2. Variation of the aggregate results in relation to case study A0 ("day work" scenario).

Impact category	Case study A0	Case study A1	Δ [%] to A0	Case study A2	Δ [%] to A0	Case study A3	Δ [%] to A0
ADP [kg Sb eq]	65 691.50	64 567.50	-1.7%	66 460.39	1.2%	53 860.14	-18.0%
AP [kg SO_2 eq]	23 349.96	22 660.97	-3.0%	23 798.08	1.9%	19 609.44	-16.0%
EP [kg PO_4^{3-}eq]	2 682.89	2 612.89	-2.6%	2 705.69	0.9%	2 263.69	-15.6%
GWP [kg CO_2 eq]	5 160 431.22	4 973 614.42	-3.6%	5 306 392.69	2.8%	4 875 633.59	-5.5%
ODP [kg CFC-11 eq]	1.11	1.10	-0.7%	1.11	0.2%	0.88	-21.0%
HTP [kg1.4-DB eq]	1 049 398.03	1 035 829.76	-1.3%	1 052 337.53	0.3%	960 218.01	-8.5%
TETP [kg 1.4-DB eq]	1 147.39	1 109.62	-3.3%	1 161.62	1.2%	1 039.55	-9.4%
POCP [kg C_2H_4]	3 872.52	3 818.10	-1.4%	3 914.52	1.1%	2 379.08	-38.6%

Finally, in relation to case study A3, major advantage is observed in comparison with case study A0. As observed from the previous sub-section, traffic congestion has an important contribution to the result of the life-cycle analysis. By enabling three lanes of traffic, the impacts due to traffic congestion are reduced, thus reducing the overall results.

6.1.1.4 Alternative maintenance scenario

In this section, an alternative maintenance plan is considered for case study A0. The alternative maintenance scenario refers to a "lack of money" situation, in which the frequency of maintenance is changed to cope with budget restrictions.

The consideration of this alternative scenario only affects the operation stage. Hence, the results of the environmental analysis for the operation stage, considering the „day work" scenario, are provided in Table 6-3 for the standard and the alternative maintenance scenarios. In the standard scenario, the results are referring to the reference case study A0.

Table 6-3. Variation of the results for the operation stage in relation to case study A0, considering "day work" scenario.

Impact category	Unit	Standard scenario	"Lack of money" scenario	Variation in relation to standard
ADP	kg Sb eq	46 916.44	37 518.38	-20.0%
AP	kg SO_2 eq	13 141.09	12 257.39	-6.7%
EP	kg PO_4^{3-} eq	1 480.66	1 623.55	9.7%
GWP	kg CO_2 eq	2 007 968.80	1 554 677.40	-22.6%
ODP	kg CFC-11 eq	0.93	0.76	-17.8%
HTP	kg1.4-DB eq	599 335.45	418 346.06	-30.2%
TETP	kg 1.4-DB eq	541.60	420.00	-22.5%
POCP	kg C_2H_4	3 081.50	2 426.58	-21.3%

The alternative maintenance plan reduces the need for maintenance actions. Consequently, the environmental impacts due to the operation stage are reduced as less material is used and traffic congestion is reduced.

6.1.2 *Life-cycle costs analysis*

6.1.2.1 Construction costs

Figure 6-3 details the construction costs for the five design solutions A0, A1, A2, A3 and A4. It is noted that (i) steel represents an important part of the total construction costs. (ii) concrete represents around a third of the total construction costs (iii) the bearing costs are insignificant when compared to other costs (iv) equipment costs are less important than those for steel and concrete.

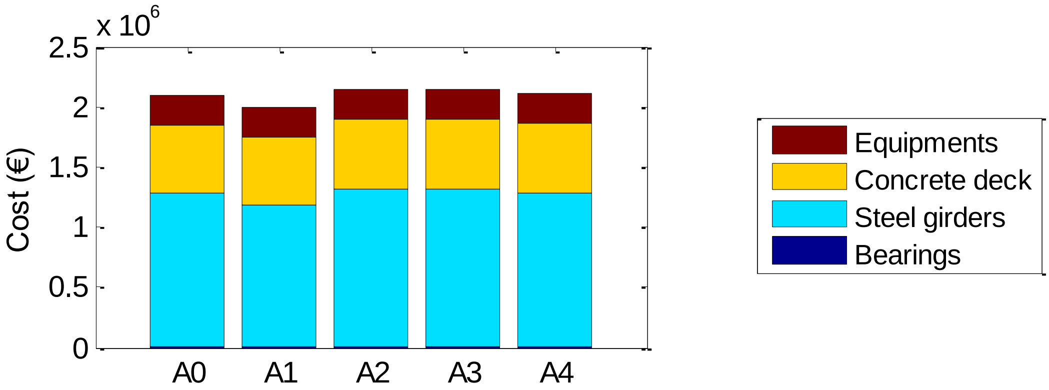


Figure 6-3. Construction costs for case studies A.

Besides, using high strength steel (S460) in design solution A1 enables to slightly reduce the quantity of steel, which explains the decrease of the construction costs for A1 (-4.5%/A0) even if the high strength steel S460 is more expensive than steel S355. It is observed that the construction costs of A2 with self-weathering steel is the most expensive one (+2.29%/A0), which can be explained by the fact that the use of self-weathering steel leads to a significant increase of steel quantities. Indeed, this solution requires an increase of thickness by 2.0 mm on each visible steel surface. A3 is more expensive than A0 at the construction stage (+2.15%/A0) due to the additional need of steel to have three traffic lanes without compromising the designed fatigue service life. For A4, a post weld treatment with high-frequent hammering is used on the welds of the stiffener webs on the lower flange at mid-span. With such a post-weld treatment, the flange thickness steel needs to be slightly increased compared to A0 but to a lesser extent as for A3, which leads to a smaller difference in construction costs between A4 and A0 (+0.36%/A0). It is noted that the construction costs of the five case studies appear relatively similar, knowing the high level of variability that can generally be observed in bridge requests for proposal.

6.1.2.2 Total life-cycle costs

Figure 6-4 compares the total LCC (construction, operation and end-of-life costs) for the different case studies A. It clearly appears in Table 6-4 that solutions that may be more expensive than others at the construction stage can finally be more attractive when considering the life-cycle of the structure. Compared with case study A0, some solutions are associated with a lower LCC (-2.8%/A0 for A1 and -4.9%/A0 for A2) and others with a larger LCC (+1.4%/A0 for A3 and +0.24%/A0 for A4). It is noted that the end-of-life costs are much lower than operation or construction costs due to the fact that these costs occur at year 100 and are discounted with a yearly discount rate fixed at 2%.

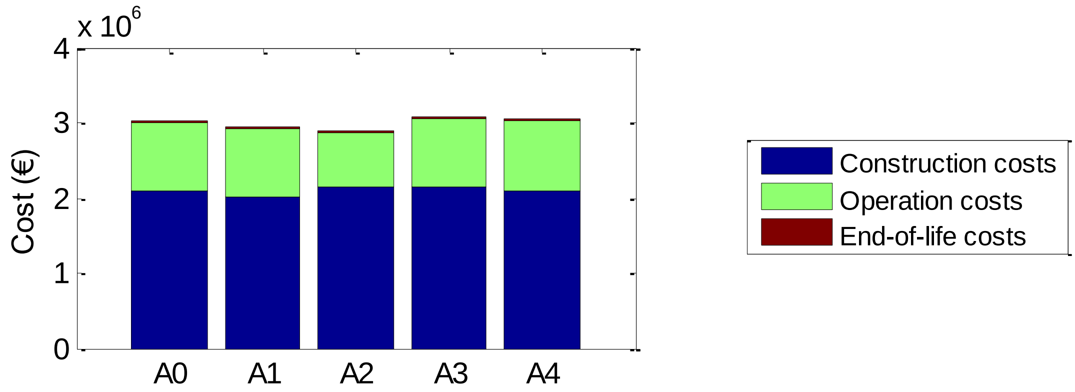


Figure 6-4. Total life-cycle costs for case studies A.

In addition to the standard maintenance scenario, a scenario with restricted financial resources is studied where the number of maintenance actions is reduced and where these actions are carried out as late as possible. For example, maintenance actions on the concrete deck at years 25, 50, 75 are replaced by only one maintenance action at year 50. Besides; maintenance action for expansion joints is only carried out at year 50 (instead of years 40 and 80 in the standard scenario). Figure 6-5 shows the life-cycle costs of case study A0 with the standard and the "lack of money" scenarios. It is observed that a lot of costly maintenance actions take place at year 50 in the "lack of money" scenario.

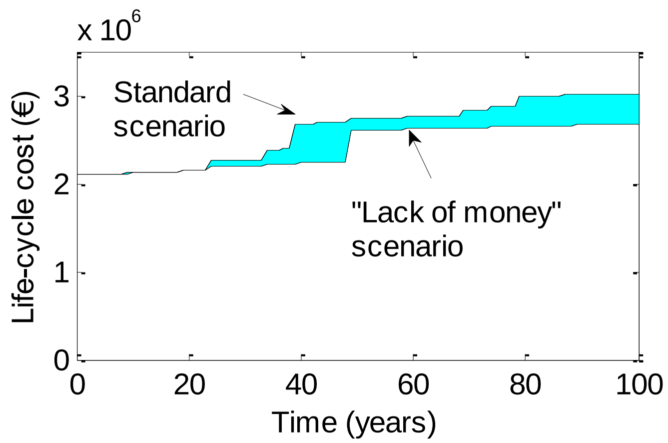


Figure 6-5. Life-cycle costs for case studies A0 with standard and "lack of money" maintenance scenarios.

6.1.3 *User costs*

Two maintenance scenarios have been studied for user costs' calculation: (i) a "day" scenario where most actions are carried out during the day (from 8 am to 5 pm) and the bridge has one lane closed for major maintenance actions (road surface/waterproofing layer replacement); (ii) "night" scenario, similar to the "day" scenario except that most of maintenance actions are carried out during the night (from 9 pm to 6 am). Figure 6-6 details the user costs for case studies A with a) "day" and (b) "night" scenario. It is noted that the user inconvenience is reduced if work is carried out during the night since there is less traffic than during the day. It is also observed that user costs for case studies A3/A4 are lower than those for A0/A1/A2 because there is a possibility of having three traffic lanes instead of two without compromising the designed fatigue service lifetime.

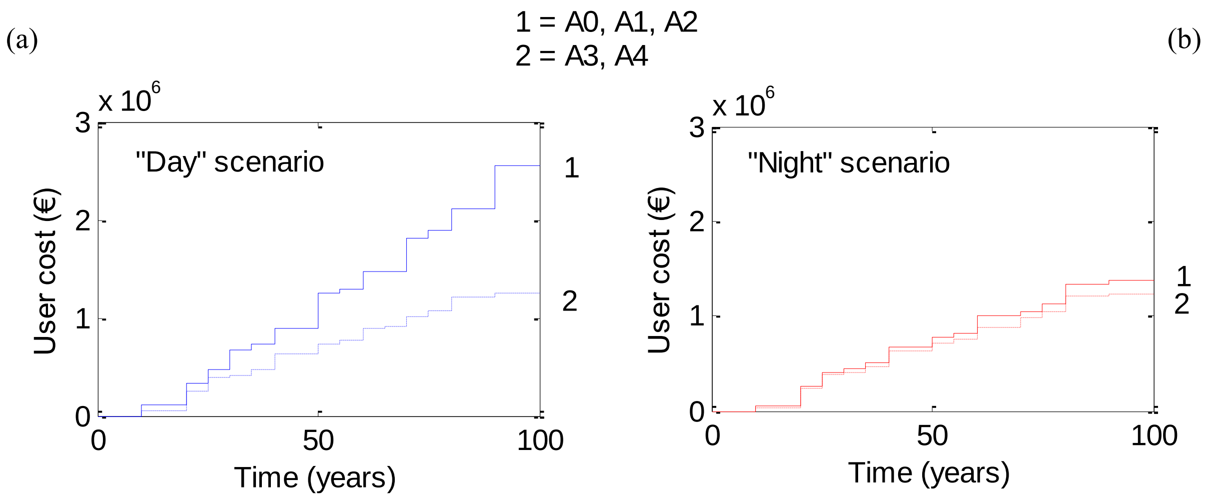


Figure 6-6. User costs for case studies A with (a) "day" and (b) "night" scenarios.

6.1.4 *Conclusions from Case A*

In this case study for small motorway bridges, it can be observed from the life-cycle environmental analysis that the stages of material production and operation are by far dominating all impact categories. In terms of processes, the production of construction materials throughout the life-cycle and traffic congestion due to work activity, are the main causes of environmental burdens in the life-cycle analysis. For the operation stage the impacts are mainly caused by traffic congestion. The overall results are improved the most by allowing three lanes of traffic as therewith congestion is reduced. The reduction of the structural steel quantity by the use of HSS and its increase by the use of self-weathering steel explains the improvement and the worsening of the results of the LCA respectively. It is observed that due to no traffic under the bridge the use of self-weathering steel has no major advantage.

The construction costs in parallel to the LCA were reduced for the use of HSS but increased for the use of self-weathering steel. Looking at total life-cycle costs the ranking was changed as the most advantageous solution of case A appears to be the application of self-weathering steel. The increase of construction costs for the three-lane section was significantly reduced by the application of post-weld treatment. User costs are reduced for the night work scenario. Both three-lane variants sum up to the least user costs.

6.2 Case B – short span bridges / motorway crossings

6.2.1 *Life-cycle environmental analysis*

6.2.1.1 Introduction

Likewise, life-cycle environmental analysis was carried out according to the methodology described in section 4.2.

The analysis takes into account all the stages, from material production to end-of-life stages, as illustrated in Figure 4-3. In all case studies, except case study B2, no traffic interference was considered for the construction and end-of-life stages. In the former case, it was assumed that being a new bridge, there was no traffic on the bridge. In the latter, it was considered that traffic would be diverted to an alternative route. In addition, due to the lack of data, the use of construction equipment was not considered in the analysis.

In the operation stage, for the quantification of environmental impacts due to traffic congestion, two alternative scenarios were considered: (i) a "day work" scenario, in which maintenance actions take place during the day (8 a.m. to 5 p.m.); and (ii) a "night work" scenario, in which maintenance actions take place during the night (9 p.m. to 6 a.m.).

In this report, only the results of the analysis considering the "day work" scenarios are provided. The detailed results of the analysis are provided in the internal reports, [18].

6.2.1.2 Reference case study B0.1

The results of the life-cycle analysis of the reference case study B0.1 are summarized in Table 6-4. For a better understanding of the contribution of each stage to the aggregated result, these results are also represented in the contribution graph Figure 6-7.

Table 6-4. Life-cycle results per life-cycle stage for B0.1.

Impact category	Unit	Total	Materials	Construction	Operation	End-of-life
ADP	kg Sb eq	9 516.43	2 438.91	12.13	7 291.19	-225.80
AP	kg SO_2 eq	3 669.15	1 702.53	5.09	2 014.39	-52.85
EP	kg PO_4^{3-} eq	400.65	162.19	0.91	211.58	25.97
GWP	kg CO_2 eq	817 912.43	572 662.64	1687.06	276 646.89	-3 3084.16
ODP	kg CFC-11 eq	0.16	0.01	0.00	0.14	0.00
HTP	kg 1.4-DB eq	192 719.97	98 034.86	241.98	87 978.38	6 464.75
TETP	kg 1.4-DB eq	190.16	102.33	0.32	81.66	5.85
POCP	kg C_2H_4	532.49	149.03	0.21	417.71	-34.46

The material production stage is the stage that most contributes to the impact category of global warming (GWP) with a percentage above 50%. On the other side, this stage has a minimum contribution to impact category ozone depletion (OD), with a percentage of about 15%. The operation stage has a major contribution to most impact categories except GWP. Likewise, it is noted that for the operation stage the processes included are the production of new materials (when applicable) and the disposal of waste materials. The environmental impacts due to the production of bridge equipment, such as bearings and expansion joints, were not considered in the analysis and no data is currently available.

Stage of construction has a negligible contribution for all impact categories; while end-of-life stage has a global contribution of less than 10%.

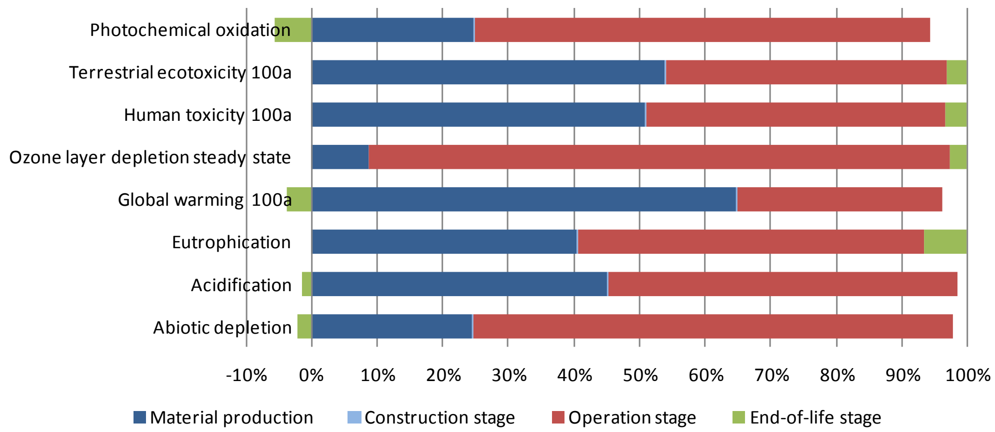


Figure 6-7. Contribution of each stage to impact category.

The results presented in Figure 6-7 were rearranged according to the main process involved in the life-cycle analysis and the results are represented in Figure 6-8. Four main processes were identified: (i) production of material, (ii) transportation of materials, (iii) disposal of materials, and (iv) traffic congestion. According to Figure 6-8, the processes of production of materials and traffic congestion are dominant for most impact categories.

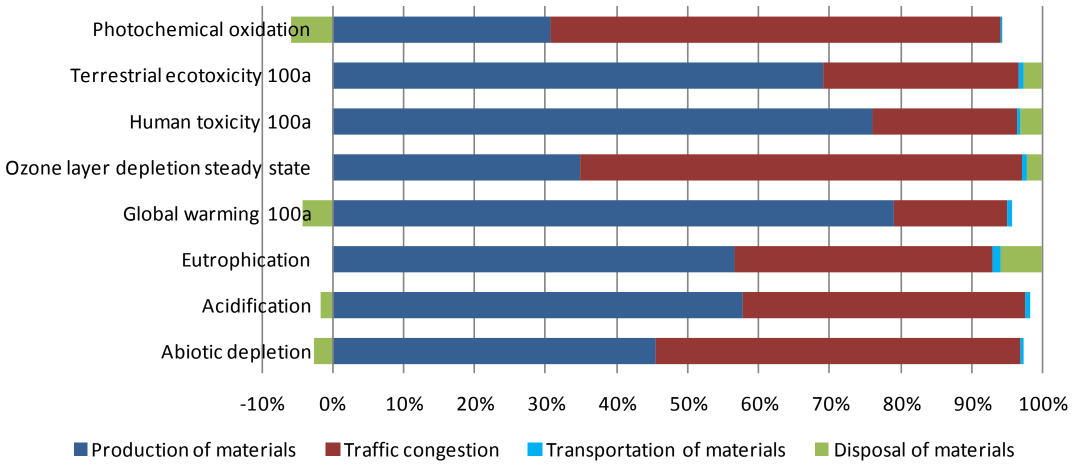


Figure 6-8. Contribution of each main process to impact category.

6.2.1.3 Aggregate life-cycle results for variants B0.2, B0.3, B0.4, B0.5, B1.1 and B1.2

The results obtained for the variant case studies B0.2, B0.3 and B0.4 are represented in Table 6-5. This table also indicates the variation of the results for each case study in relation to the reference case study B0.1. As the bridges have different spans and deck widths, the results are provided by the area of each deck.

Table 6-5. Variation of the aggregate results, in m^2, in relation to case study B0.1.

Impact category	Unit	Case study B0.1	Case study B0.2	Δ to B0.1	Case study B0.3	Δ to B0.1	Case study B0.4	Δ to B0.1
ADP	kg Sb eq/m^2	17.56	14.71	-16.3%	16.66	-5.1%	17.69	0.7%
AP	kg SO_2 eq/m^2	6.77	5.93	-12.4%	6.46	-4.6%	6.64	-1.9%
EP	kg PO_4^{3-} eq/m^2	0.74	0.67	-9.0%	0.73	-1.9%	0.72	-2.6%
GWP	kg CO_2 eq/m^2	1 509.62	1 418.58	-6.0%	1 429.58	-5.3%	1395.92	-7.5%
ODP	kg CFC-11 eq/m^2	0.00	0.00	-16.2%	0.00	-3.1%	0.00	2.9%
HTP	kg1.4-DB eq/m^2	355.70	364.89	2.6%	378.87	6.5%	335.42	-5.7%
TETP	kg 1.4-DB eq/m^2	0.35	0.33	-6.7%	0.34	-3.6%	0.34	-4.2%
POCP	kg C_2H_4/m^2	0.98	0.80	-18.4%	0.99	0.4%	1.05	7.3%

Likewise, the results obtained for the variant case studies B0.5, B1.1 and B1.2 are represented in Table 6-6. This table also indicates the variation of the results for each case study in relation to the reference case study B0.1.

Most case studies variants have advantages in relation to case study B0.1, particularly case study B0.2 with reductions higher than 10% for some impact categories. For the other case studies, except case study B1.2, the results are similar, with lower results in some environmental categories and higher in others. However, the differences are less than 10% in each case.

By avoiding expansion joints, case studies B1.1 and B1.2 have advantage in relation to the remaining case studies, particularly in relation to the operation stage, as less maintenance actions are needed. However, considering the complete life cycle of the bridge, case study B1.2 has no advantages in relation to the reference case study and this is mainly due to the use of massive materials in the foundations, which globally represents a higher environmental burden.

Table 6-6. Variation of the aggregate results, in m^2, in relation to case study B0.1.

Impact category	Unit	Case study B0.1	Case study B0.5	Δ to B0.1	Case study B1.1	Δ to B0.1	Case study B1.2	Δ to B0.1
ADP	kg Sb eq/m^2	17.56	17.32	-1.4%	17.46	-0.6%	19.21	9.4%
AP	kg SO_2 eq/m^2	6.77	6.50	-4.1%	6.58	-2.9%	7.24	6.9%
EP	kg PO_4^{3-} eq/m^2	0.74	0.71	-4.4%	0.72	-2.7%	0.74	-0.2%
GWP	kg CO_2 eq/m^2	1 509.62	1 369.96	-9.3%	1 467.42	-2.8%	1 601.65	6.1%
ODP	kg CFC-11 eq/m^2	0.00	0.00	1.1%	0.00	-1.2%	0.00	-1.0%
HTP	kg1.4-DB eq/m^2	355.70	332.25	-6.6%	336.71	-5.3%	285.09	-19.9%
TETP	kg 1.4-DB eq/m^2	0.35	0.33	-6.2%	0.34	-2.5%	0.36	3.9%
POCP	kg C_2H_4/m^2	0.98	1.03	5.0%	0.94	-4.1%	1.06	7.6%

6.2.1.4 Alternative maintenance scenario

In this section, an alternative maintenance plan is considered for case study B0.1. The alternative maintenance scenario refers to a "lack of money" situation, in which the frequency of maintenance is change to cope with budget restrictions.

The consideration of this alternative scenario only affects the operation stage. Hence, the results of the environmental analysis for the operation stage, considering the „day work" scenario, are provided in Table 6-7 for the standard and the alternative maintenance scenarios. In the standard scenario, the results are referring to the reference case study B0.1.

Table 6-7. Variation of the results for the operation stage in relation to case study B0.1, considering "day work" scenario.

Impact category	Unit	Standard scenario	"Lack of money" scenario	Variation in relation to standard
ADP	kg Sb eq	13.46	7.27	-46.0%
AP	kg SO_2 eq	3.72	1.96	-47.2%
EP	kg PO_4^{3-} eq	0.39	0.21	-45.7%
GWP	kg CO_2 eq	510.61	300.34	-41.2%
ODP	kg CFC-11 eq	0.00	0.00	-43.8%
HTP	kg1.4-DB eq	162.38	86.56	-46.7%
TETP	kg 1.4-DB eq	0.15	0.08	-45.8%
POCP	kg C_2H_4	0.77	0.40	-47.6%

Similarly to case study A0, the alternative maintenance plan reduces the need for maintenance actions. Consequently, the environmental impacts due to the operation stage are reduced as less material is used and traffic congestion is reduced, both above and below the bridge.

6.2.1.5 Reference case studies B2.1 and B2.2

Case study B2.1 takes into account all the stages indicated in Figure 4-3. Contrary to the remaining case studies, in this case, traffic congestion due to the stages of constructions and end-of-life are taken into account. In addition, the burdens due to the use of equipment are considered in all stages over the life-cycle of the bridge.

The results of the life-cycle analysis of case study B2.1 are summarized in Table 6-8. For a better understanding of the contribution of each stage to the aggregated result, these results are also represented in the contribution graph of Figure 6-9.

Table 6-8. Life-cycle results per life-cycle stage B2.1.

Impact category	Unit	Total	Materials	End-of-life	Construction	Operation
ADP	kg Sb eq	16 936.01	3617.49	876.21	2 164.33	10 277.97
AP	kg SO_2 eq	7 610.91	2 472.37	425.06	830.29	3 883.19
EP	kg PO_4^{3-} eq	950.28	219.09	88.04	112.56	530.59
GWP	kg CO_2 eq	1 492 913.24	874 943.06	-68 079.83	111 361.63	574 688.38
ODP	kg CFC-11 eq	0.30	0.01	0.03	0.04	0.21
HTP	kg 1.4-DB eq	219 296.30	96 349.93	18 740.04	18 488.88	85 717.45
TETP	kg 1.4-DB eq	301.23	140.45	26.46	24.23	110.08
POCP	kg C_2H_4	1 172.28	251.87	39.70	194.62	686.09

The material production stage is the stage that most contributes to the impact category of global warming (GWP) with a percentage close to 50%. On the other side, this stage has a minimum contribution to impact category ozone depletion (OD), with a percentage lower than 5%. The operation stage has a significant contribution to most impact categories except GWP.

Stage of construction has a contribution of about 10% for all impact categories; while end-of-life stage has a global contribution of less than 10% for most impact categories.

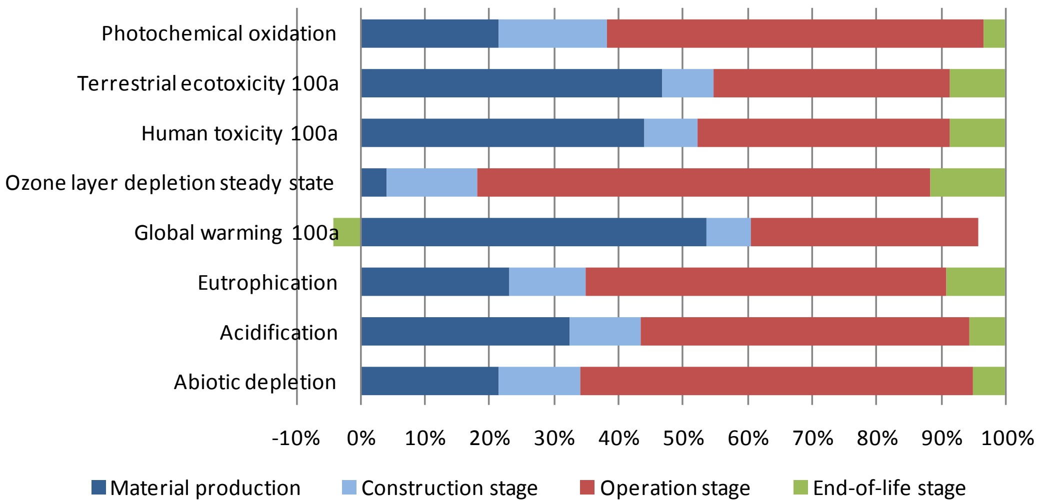


Figure 6-9. Contribution of each stage to impact category.

The results presented in Figure 6-9 were rearranged according to the main process involved in the life-cycle analysis and the results are represented in Figure 6-10. In this case, five main processes were identified: (i) production of material, (ii) transportation of materials, (iii) use of equipment, (iv) disposal of materials, and (v) traffic congestion. According to Figure 6-10, the processes of production of materials and traffic congestion are dominant for most impact categories. The use of equipment has a significant contribution to the impact categories of global warming, acidification and eutrophication. Finally, the transportation of materials has a negligible contribution to the global results.

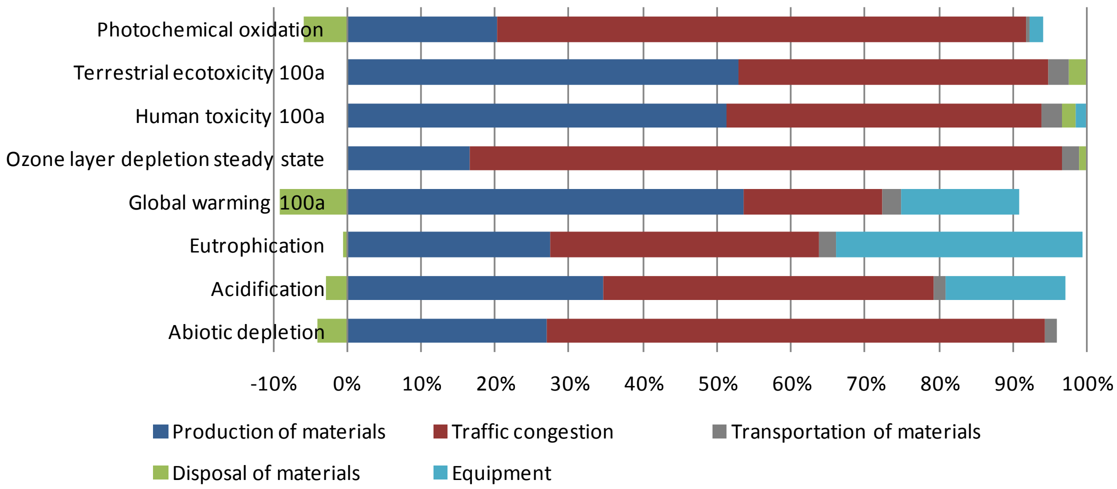


Figure 6-10. Contribution of each main process to impact category ("day work" scenario).

Case study B2.2 is a variant of case study B2.1. The only difference between case study B2.1 and B2.2 is the use of self-weathering steel in the main structure of the latter.

Comparing case study B2.1 to case study B2.2, the results are summarized in Table 6-9.

Table 6-9. Variation of the aggregate results of case study B2.2 in relation to case study B2.1.

Impact category	Unit	Case study B2.1	Case study B2.2	Variation in relation to B2.1
ADP	kg Sb eq	16 936.01	15 020.73	-11.3%
AP	kg SO_2 eq	7 610.91	6 948.33	-8.7%
EP	kg PO_4^{3-} eq	950.28	862.24	-9.3%
GWP	kg CO_2 eq	1 492 913.24	1 428 818.88	-4.3%
ODP	kg CFC-11 eq	0.30	0.26	-13.1%
HTP	kg1.4-DB eq	219 296.30	203 006.69	-7.4%
TETP	kg 1.4-DB eq	301.23	280.52	-6.9%
POCP	kg C_2H_4	1 172.28	1 046.83	-10.7%

In this case, the use of self-weathering steel has clear advantages in relation to the standard coating system, as the maintenance of the steel structure is avoided and consequently, the environmental burdens due traffic congestion under the bridge are reduced.

6.2.2 *Life-cycle costs analysis*

6.2.2.1 Construction costs

The construction costs in € and in €/m^2 for nine design solutions are presented in Figure 6-11 a and b, respectively.

For concrete bridges B0-2 (-1.89%/B0-1) and B0-3 (+12.52%/B0-1), it is obviously noted that concrete represents a significant part of the construction costs. B0-3 is more expensive than others due to the high costs of prefabricated prestressed beams.

It is also noted that the construction costs for B0-4 and B0-5 are slightly lower than that for B0-1 (-2.19%/B0-1 and -4.82%/B0-1, respectively), mainly since plated steel (B0-1) is more expensive than rolled steel (B0-4/B0-5). Also, the solution B0-5 with hot rolled steel S460 enables steel quantity reduction and turns out to be less expensive than B0-4.

For case studies B1-1 (-13.35%/B0-1) and B1-2 (14.34%/B0-1), it is observed that the design of abutments may strongly impact the construction costs. In particular, the foundations by sheet piling are more expensive than those in concrete in B1-1. However, this type of foundations enables to reduce the work duration since it does not require the concreting phase, which may take a lot of time. Between these two integral bridges, B1-2 is more expensive than B1-1 not only for foundations but also in steel girders costs. Besides, hot rolled steel girders do not require welded sections and can resist to fatigue problem in a better way than welded sections with plated steel. Unfortunately, this advantage is not quantified in this study since no maintenance actions associated with fatigue problem of steel girder are considered.

Case studies B2 are the most expensive ones since they are the longest ones. It is due to their design with two spans as counter-weight for the middle span. This design allows building the bridge mostly on the sides of the highway underneath and having less traffic disruption. Similar to case study A2 (section 6.1.2) case study B2-2 with self-weathering steel is more expensive than B2-1 in term of construction costs.

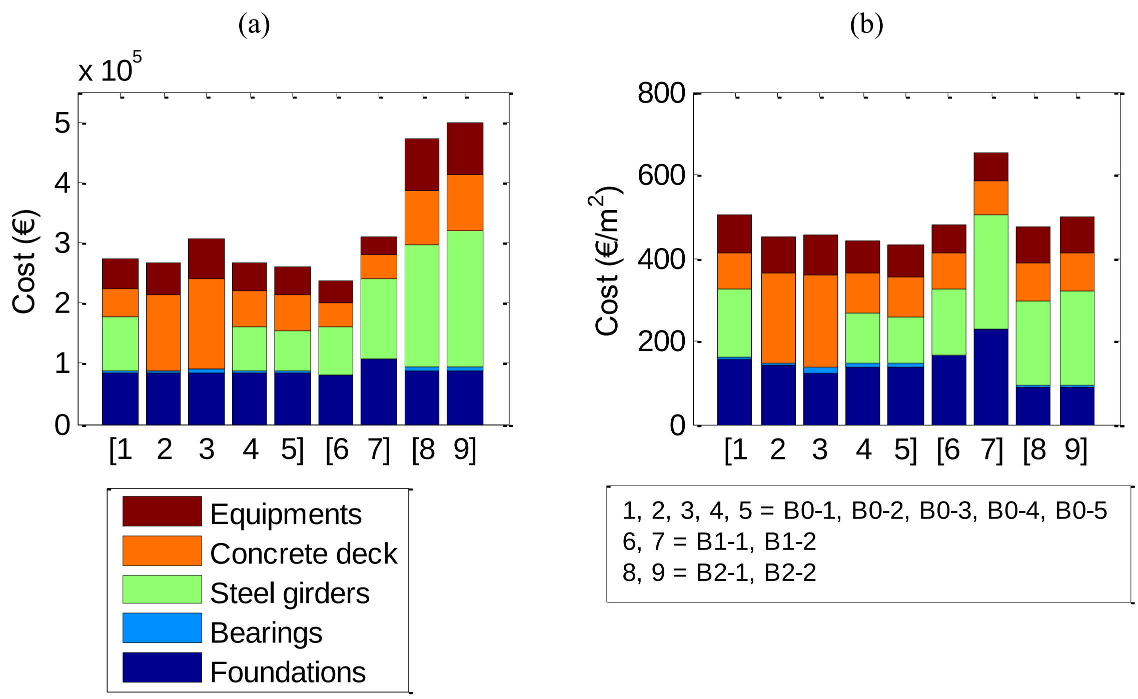


Figure 6-11. Construction costs for case studies B in (a) € and (b) in €/m².

6.2.2.2 Total life-cycle costs

The total life-cycle costs in € and in €/m^2 are presented in Figure 6-12 a and b, respectively.

Compared with the total life-cycle costs (construction/operation and end-of-life costs) of case study B0-1, some design solutions associated with lower total LCC (-6.24%/B0-1 for B0-2. -1.54%/B0-1 for B0-5 and -14.61%/B0-1 for B1-1) and others with larger LCC (7.76%/B0-1 for B0-3. 0.47%/B0-1 for B0-4 and 0.86%/B0-1 for B1-2).

It is noted that B0-2 and B0-3 have lower operation costs than B0-1 due to the fact that concrete bridges do not require the maintenance actions for corrosion protection. However, it is reminded that they have more concrete surface to maintain.

No significant difference is noticed between case studies with hot rolled steel girders (B0-4. B0-5) with B0-1 (plated steel girders) since maintenance actions are similar for both types.

It is observed integral bridges (B1-1 and B1-2) allow to significantly reduce operation costs due to the lack of maintenance actions concerning expansion joints.

When comparing B2-1 with B2-2, case study B2-2 with self-weathering steel allows significantly reducing the maintenance costs due to the lack of costly maintenance actions concerning to corrosion protection.

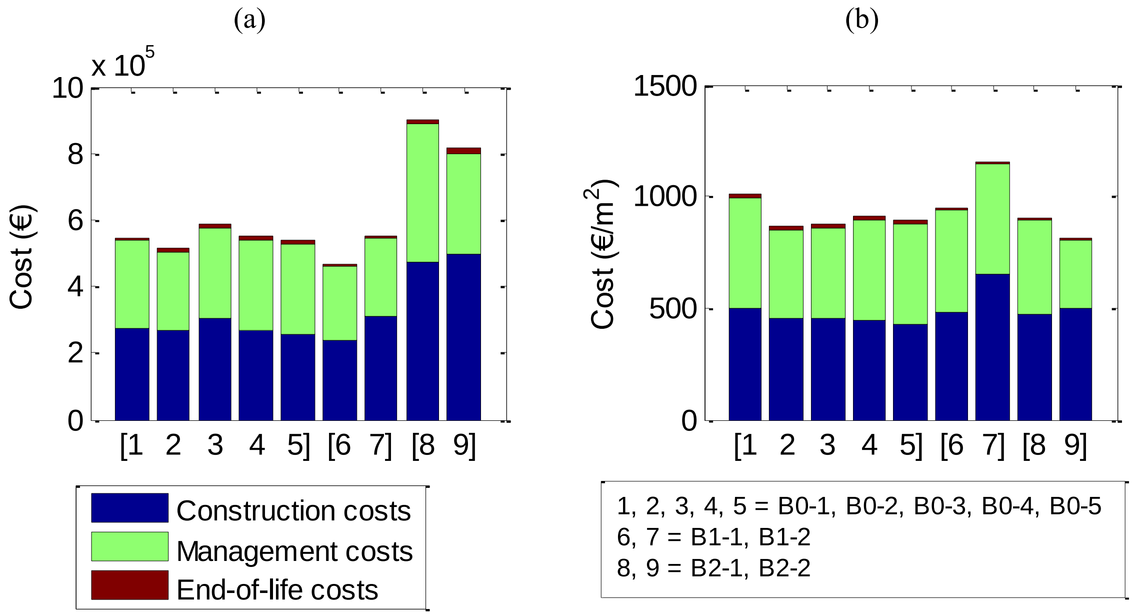


Figure 6-12. Total life-cycle costs for case studies B in (a) € and (b) in €/m².

Similar to case studies A (section 6.1.2), a scenario with restricted financial resources is studied for the case study B0-1, Figure 6-13.

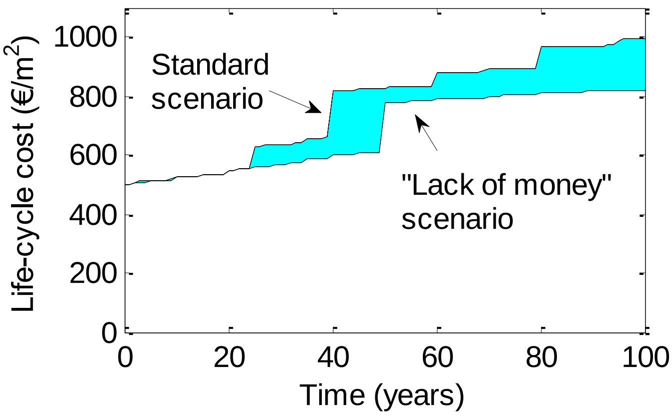


Figure 6-13. Life-cycle costs for case study B0-1 with 2 maintenance scenarios.

6.2.3 *User costs*

Similar to case A (compare section 6.1.3), two scenarios ("day" and "night") are applied for user costs' calculation. The user costs for "day" and "night" scenarios are presented in Figure 6-14 a and b, respectively. It is observed that the integral bridges require fewer maintenance actions due to their design and, consequently, lead to lower traffic disruption level during the lifetime. There is a relatively small difference between B0-1 and B0-4/B0-5 since maintenance actions of plated and hot rolled steel girders are similar. Besides, the user costs are higher for case studies B0-2 and B0-3 because they have more concrete surface to maintain, which requires traffic disruptions of the highway underneath the bridge. Finally, case studies B2 have large user costs since the quantities of elements to be maintained are larger than those for other B case studies.

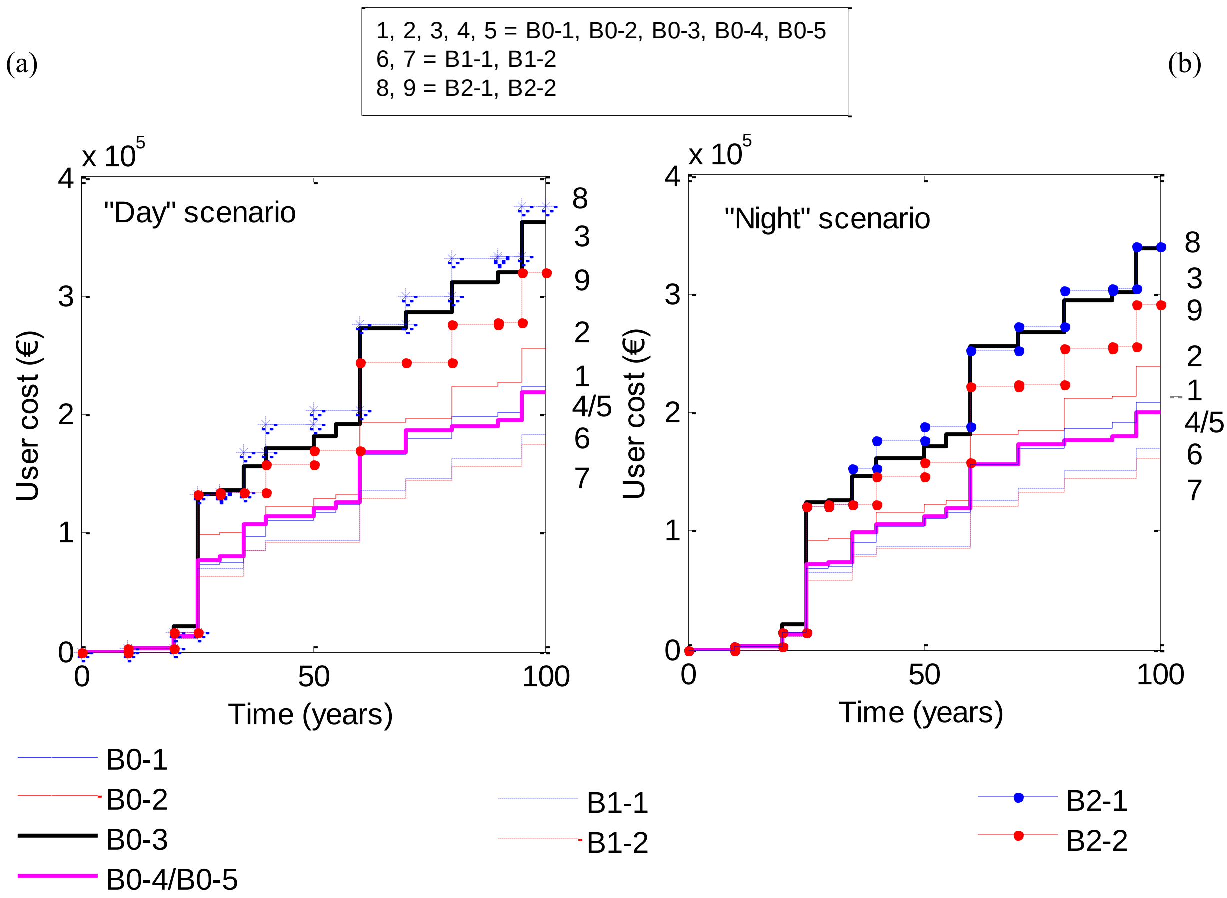


Figure 6-14. User costs for case study B (a) – "day" scenario and (b) – "night" scenario.

6.2.4 *Conclusions from Case B*

For crossings of motorways the environmental impacts of the material production and the operation stage dominate by far the life-cycle. In general a reduction of these impacts is achieved as well for integral bridge solutions as for three-span bridges. Also the concrete variants B0-2 and B-3 show advantages. Taking the use of equipment into account a significant contribution is noticed to GW, AP and EP, whereas the transportation of materials is negligible. For crossings of a motorway a big benefit was registered for the use of self-weathering steel as the maintenance of any coating is avoided and therewith traffic congestion reduced.

Looking at the construction costs especially for integral solutions the foundations play a decisive role. In total the three-span solutions are more expensive, being relativized if taking the increased bridge length into account. The integral bridges significantly reduce operating costs due to the lack of maintenance actions concerning expansion joints. A similar reduction is noted for the use of self-weathering steel significantly reducing the maintenance costs due to the lack of costly maintenance actions concerning to corrosion protection. The design of integral bridges requires less maintenance and therefore leads to less traffic disruption and reduced user costs. The three-span bridges on the other extreme have large user costs as a higher quantity of elements requires maintenance and therefore traffic disruption is caused.

It is noted that the concrete alternatives B0-2 and B0-3 have lower operation costs than the standard steel-composite solution B0-1 due to the fact that concrete bridges do not require the maintenance actions for corrosion protection. However, they have more concrete surface to maintain which requires traffic disruptions of the highway underneath the bridge and causes higher user costs.

6.3 Case C – long span bridges

6.3.1 *Life-cycle environmental analysis*

6.3.1.1 Introduction

Like in previous case studies, life-cycle environmental analysis was carried out according to the methodology described in section 4.2. In this case study only the superstructure is considered in the life-cycle analyses without taking any pillars or foundations into account.

The analysis takes into account all the stages, from material production to end-of-life stages, as illustrated in Figure 4-3. However, no traffic interference was considered for the construction and end-of-life stages. In the former case, it was assumed that being a new bridge, there was no traffic on the bridge. In the latter, it was considered that traffic would be diverted to an alternative route. In addition, due to the lack of data, the use of construction equipment was not considered in the analysis.

In the operation stage, for the quantification of environmental impacts due to traffic congestion, two alternative scenarios were considered: (i) a "day work" scenario, in which maintenance actions take place during the day (8 a.m. to 5 p.m.); and (ii) a "night work" scenario, in which maintenance actions take place during the night (9 p.m. to 6 a.m.).

In this report, only the results of the analysis considering the "day work" scenarios are provided. The detailed results of the analysis are provided in the internal reports, [19].

6.3.1.2 Reference case study C0

The results of the life-cycle analysis of the reference case study C0 are summarized in Table 6-10. For a better understanding of the contribution of each stage to the aggregated result, these results are also represented in the contribution graph of Figure 6-15.

Table 6-10. Life-cycle results per life-cycle stage.

Impact category	Unit	Total	Materials	Construc-tion	Operation	End-of-life
ADP	kg Sb eq	334 083.11	126 152.31	487.79	240 491.83	-33 048.82
AP	kg SO_2 eq	134 475.76	66 673.83	204.52	78 811.84	-11 214.43
EP	kg PO_4^{3-} eq	15 959.42	6 390.17	36.47	9 746.79	-214.01
GWP	kg CO_2 eq	26 399 001.73	22 640 957.00	67 839.93	10 038 699.00	-6 348 494.20
ODP	kg CFC-11 eq	5.49	0.75	0.01	4.68	0.05
HTP	kg 1.4-DB eq	4 644 555.78	1 920 156.80	9 730.67	2 610 489.70	104 178.61
TETP	kg 1.4-DB eq	5 872.89	2 931.40	12.92	2 649.72	278.85
POCP	kg C_2H_4	41 669.59	7 469.16	8.47	37 450.57	-3 258.62

The material production stage is the stage that most contributes to the impact category of global warming (GWP) with a percentage above 50%. On the other side, this stage has a minimum contribution to impact category ozone depletion (Ph), with a percentage of less than 15%. The operation stage has a major contribution to most impact categories except GWP.

Stage of construction has a negligible contribution for all impact categories; while end-of-life stage has a global contribution of less than 10%.

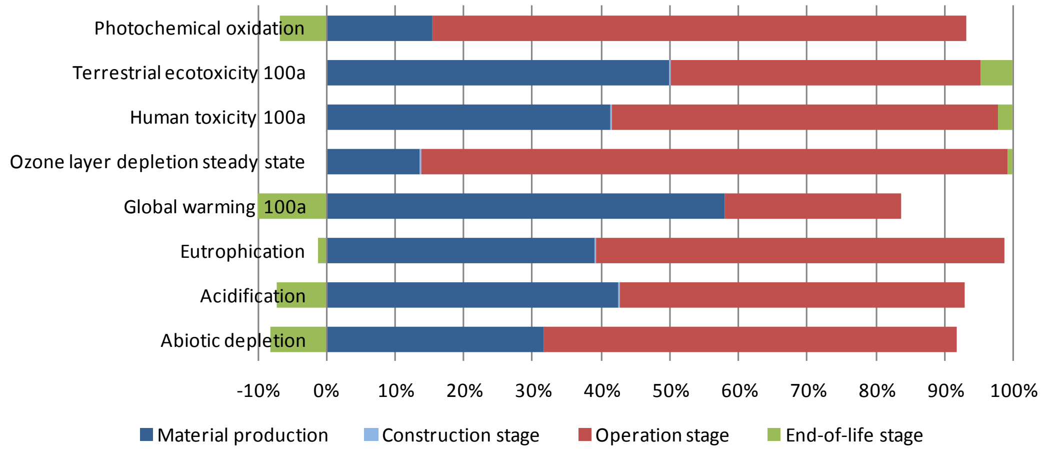


Figure 6-15. Contribution of each stage to impact category.

The results presented in the previous figure were rearranged according to the main process involved in the life-cycle analysis and the results are represented in Figure 6-16. Four main processes were identified: (i) production of material, (ii) transportation of materials, (iii) disposal of materials, and (iv) traffic congestion. According to Figure 6-16, the processes of production of materials and traffic congestion are dominant for most impact categories.

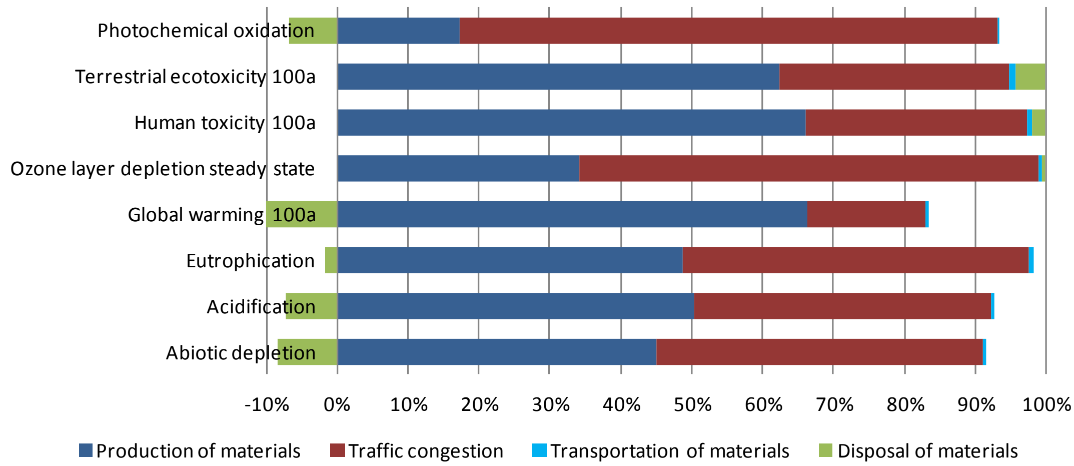


Figure 6-16. Contribution of each main process to impact category.

6.3.1.3 Aggregate life-cycle results for variants C1.1 and C1.2

The results obtained for the variant case studies C1.1 and C1.2 are represented in Table 6-11. This table also indicates the variation of the results for each case study in relation to the reference case study C0.

Both case studies variants C1.1 and C1.2 have advantage in relation to case study C0. Due to the use of HSS, case study C1.2 enables to reduce the amount of structural steel and therefore to reduce the environmental impacts due to the construction stage.

In relation to case study C1.1, major advantage is observed in comparison with case study C0. As already referred, traffic congestion has an important contribution to the result of the life-cycle analysis. Being a two-deck bridge, it is possible to divert the traffic during maintenance actions from one deck to the other, enabling two lanes of traffic in each direction. Hence, the impacts due to traffic congestion are reduced, thus reducing the overall results.

Table 6-11. Variation of the aggregate results in relation to case study C0.

Impact category	Unit	Case study C0	Case study C1.1	Variation in relation to C0	Case study C1.2	Variation in relation to C0
ADP	kg Sb eq	334 083.11	219 928.40	-34.2%	318 973.62	-4.5%
AP	kg SO_2 eq	134 475.76	95 078.74	-29.3%	125 213.43	-6.9%
EP	kg PO_4^{3-} eq	15 959.42	10 675.29	-33.1%	15 017.96	-5.9%
GWP	kg CO_2 eq	26 399 001.73	23 656 455.44	-10.4%	23 888 256.46	-9.5%
ODP	kg CFC-11 eq	5.49	3.17	-42.3%	5.38	-1.9%
HTP	kg1.4-DB eq	4 644 555.78	3 869 577.34	-16.7%	4 461 972.70	-3.9%
TETP	kg 1.4-DB eq	5 872.89	5 010.17	-14.7%	5 365.01	-8.6%
POCP	kg C_2H_4	41 669.59	12 694.40	-69.5%	40 938.49	-1.8%

6.3.1.4 Alternative maintenance scenario

In this section, two alternative maintenance plans are considered for case study C0. As for the remaining case studies, the first alternative maintenance scenario refers to a “lack of money” situation, in which the frequency of maintenance is change to cope with budget restrictions.

The second alternative maintenance scenario refers to the “prolonged life” situation, in which the service life of the bridge is extended to 130 years.

Both alternative scenarios only affect the operation stage. Hence, the results presented in this section refer only to the operation stage.

The results of the environmental analysis for the operation stage, considering the „day work“ scenario, are provided in Table 6-12 for the standard and both alternative maintenance scenarios. In the standard scenario, the results are referring to the reference case study C0.

Table 6-12. Variation of the results for the operation stage in relation to case study C0, considering “day work” scenario.

Impact category	Unit	Standard scenario	“Lack of money” scenario	Δ to C0	“Prolonged life” scenario	Δ to C0
ADP	kg Sb eq	240 491.83	250 470.36	4.1%	535 386.17	122.6%
AP	kg SO_2 eq	78 811.84	84 091.29	6.7%	180 745.27	129.3%
EP	kg PO_4^{3-} eq	9 746.79	10 494.78	7.7%	22 401.36	129.8%
GWP	kg CO_2 eq	10 038 699.00	10 361 425.00	3.2%	21 792 689.00	117.1%
ODP	kg CFC-11 eq	4.68	4.97	6.2%	10.56	125.6%
HTP	kg1.4-DB eq	2 610 489.70	2 355 219.70	-9.8%	4 997 587.70	91.4%
TETP	kg 1.4-DB eq	2 649.72	2 686.64	1.4%	5 745.78	116.8%
POCP	kg C_2H_4	37 450.57	43 852.26	17.1%	93 877.70	150.7%

As the two alternative maintenance scenarios refer to two different time spans, the results of the environmental analysis for the operation stage, considering the „day work“ scenario, are provided in Table 6-13 per year. A time span of 100 years and 130 years were considered for the “lack of money” scenario and the “prolonged life” scenarios, respectively.

Table 6-13. Variation of the results for the operation stage in relation to case study C0,. considering "day work" scenario (per year).

Impact category	Unit	Standard scenario	"Lack of money" scenario	Δ to C0	"Prolonged life" scenario	Δ to C0
ADP	kg Sb eq/yr	2404.91	2504.70	4.1%	4118.36	71.2%
AP	kg SO_2 eq/yr	788.11	840.91	6.7%	1390.35	76.4%
EP	kg PO_4^{3-} eq/yr	97.46	104.94	7.7%	172.32	76.8%
GWP	kg CO_2 eq/yr	100386.99	103614.25	3.2%	167636.07	67.0%
ODP	kg CFC-11 eq/yr	0.04	0.04	6.2%	0.08	73.6%
HTP	kg1.4-DB eq/yr	26104.89	23552.19	-9.8%	38442.98	47.3%
TETP	kg 1.4-DB eq/yr	26.49	26.86	1.4%	44.20	66.8%
POCP	kg C_2H_4/yr	374.50	438.52	17.1%	722.14	92.8%

Contrary to the previous case studies, the "lack of money" scenario slightly increases the results of most impact categories, as major maintenance actions take place in years 70 and 90, in which the average daily traffic is too high.

In relation to the "prolonged life" scenario, the effort to prolong the service life of the bridge corresponds to an increase of all environmental categories.

6.3.2 *Life-cycle costs analysis*

6.3.2.1 *Construction costs*

The total construction costs C_{cons} for design solutions C0, C1-1, and C1-2 are assessed at 20.5 M€. 24.3 M€ (+18.8% / C0) and 19.4 M€ (-5.4% / C0), respectively. Figure 6-17 shows that (i) construction costs for C1-2 with high strength steel (steel S355, S460 and S690) is lower than those for C0 (steel S355) and C1-1 (steel S355), the increase in price for high strength steel being balanced by the lower quantities of material, (ii) the case C1-1 with two bridge decks is obviously more expensive than C0 and C1-2 mainly due to the fact that larger quantities of materials are needed to build two bridge decks, (iii) steel obviously represents the major part of the construction costs for composite bridges.

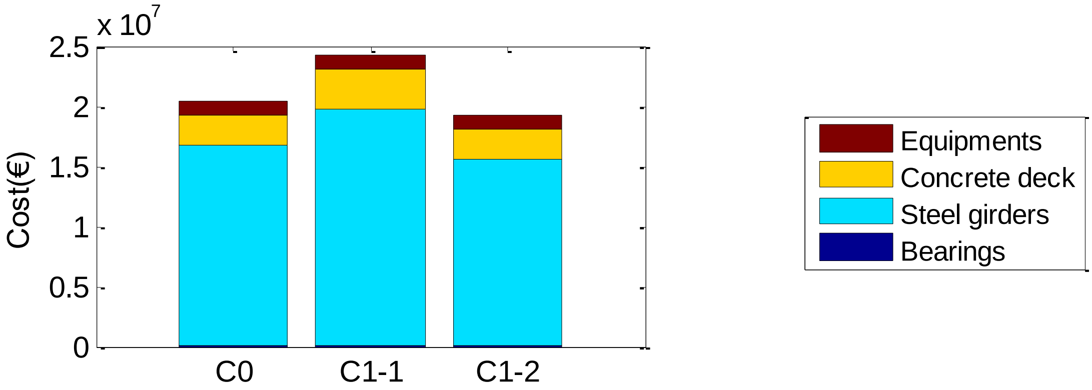


Figure 6-17. Construction costs for case studies C.

6.3.2.2 *Total life-cycle costs*

A comparison of the total LCC including construction, operation, and end-of-life actions is finally proposed in Figure 6-18. It is noted that the maintenance scenario is the same for the cases studies C. The hierarchical order remains then the same than the one obtained at the construction stage.

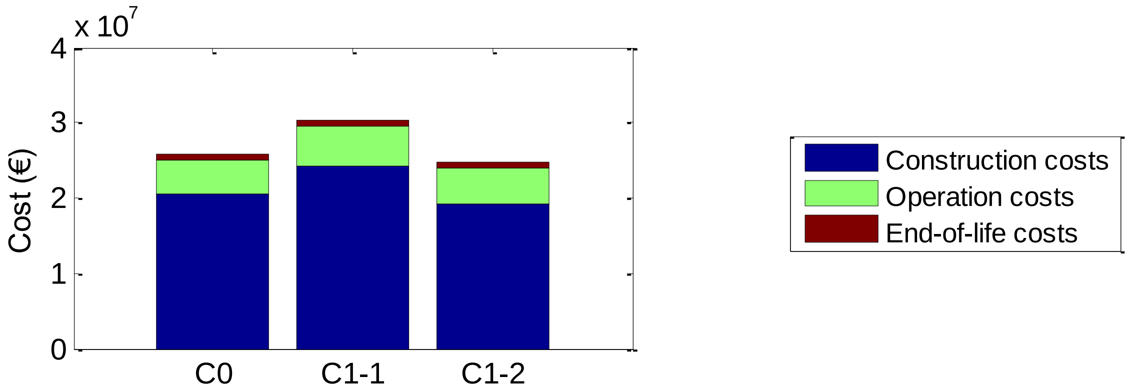


Figure 6-18. Total life-cycle costs for case studies C.

Besides, a scenario with prolonged life is studied, where a decision is made at year 80 to keep the bridge in service longer than the designed service life (130 years instead of 100). Maintenance actions strategy is adapted at the end of service lifetime to ensure an adequate level of performance of the bridges until year 130. Figure 6-19 shows the life-cycle costs for case study C0 with standard and "prolonged life" maintenance scenarios. It is noted that the rate of increase of life-cycle costs is lower after year 80 than that at the beginning of the service life since costs are discounted with a fixed yearly discount rate of 2%.

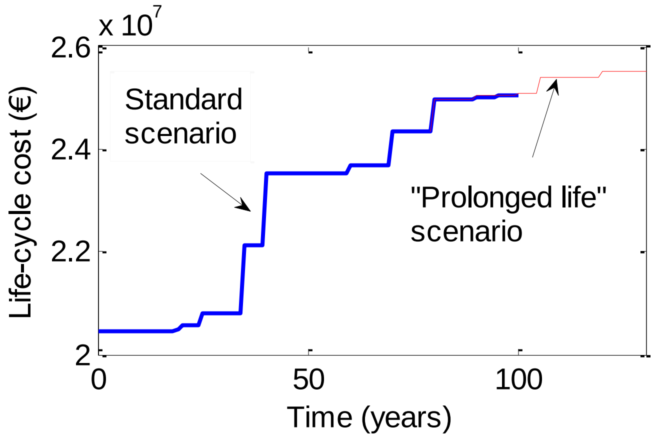


Figure 6-19. Life-cycle costs for case study C0 with standard and "prolonged life" maintenance scenarios.

6.3.3 *User costs*

Similar to case studies A and B (compare section 6.1.3 and section 6.2.3), two scenarios are applied for case studies C, Figure 6-20. It is noted that the case study C1-1 with 2 bridge deck offers the possibility to have 3 lanes in one deck to divert the traffic on the other one in order to reduce the traffic jams. Case studies C0 and C1-2 cannot offer such a solution since they do not have enough space to do so.

(a)

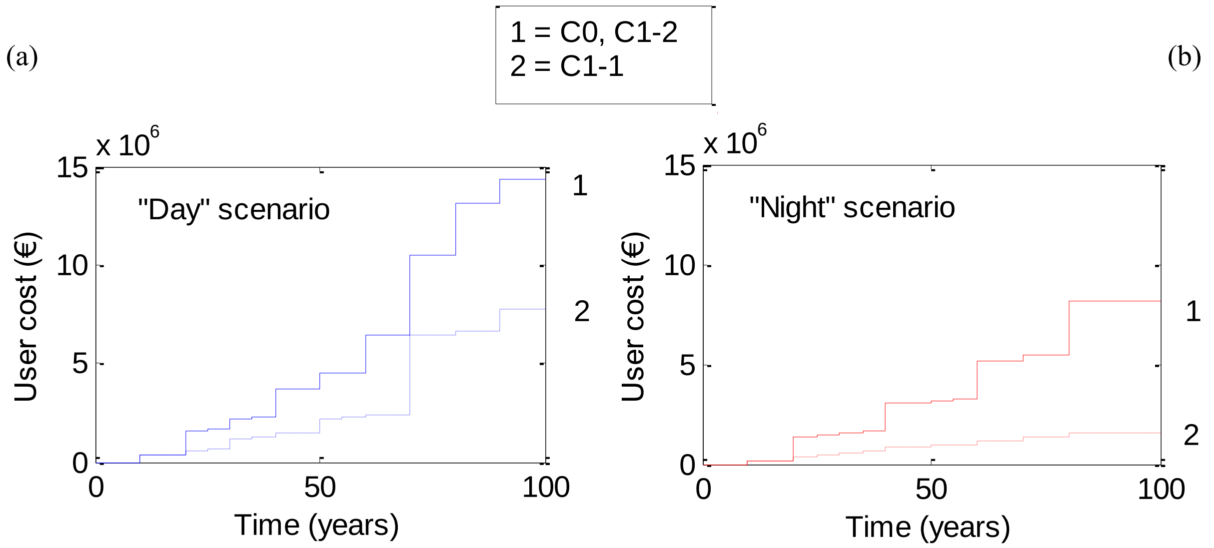


Figure 6-20. User costs for case studies (a) C0/C1-2 and (b) C1-1.

Similar to section 6.1.3, section 6.2.3 and in harmony with section 6.3.2 (Figure 6-19), the user costs of the case study C0 with standard and "prolonged life" scenarios are presented in Figure 6-21.

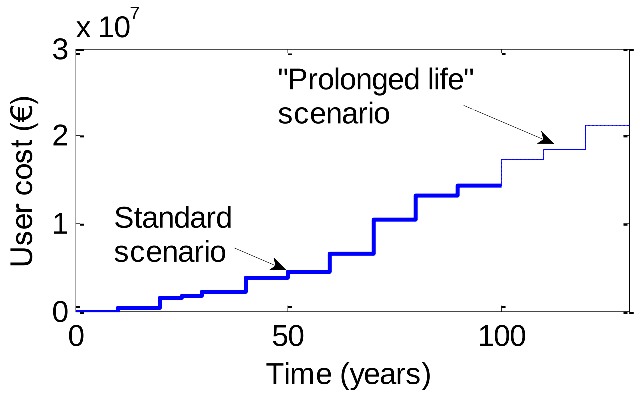


Figure 6-21. User costs for case study C0 - scenario "prolonged life".

6.3.4 *Conclusions from Case C*

Similar conclusions on the life-cycle environmental analysis can be drawn for the big motorway bridges, as before again the material production and the operation stage dominate the life-cycle. For the Global warming the input from the traffic congestion is not as big as for the other categories where as for the Photochemical oxidation the production of materials contributes by far less than the traffic congestion. For the two-deck bridge it is possible to significantly reduce traffic congestion as traffic is diverted during maintenance actions. A reasonable improvement of the environmental impacts was achieved by the use of HSS. Contrary to the previous case studies, the "lack of money" scenario slightly increases the results of most impact categories, as major maintenance actions take place in years 70 and 90.

Regarding the life-cycle costs, the variant using HSS allows having the lowest construction costs and therefore for smallest total costs. The two-deck solution on the one hand is the most costly one regarding life-cycle costs but on the other hand reduces user costs significantly.

6.4 Multi-criteria approach

6.4.1 *Introduction*

The life-cycle approach proposed in the scope of this project aimed for the integration of different criteria in the context of sustainability. To fulfil the aim of the proposed approach, outranking based methods are preferred to aggregating methods (or single criterion methods) because they involve weaker trade-offs, [137].

The method adopted in this research project is the Preference Ranking Organization Methodology of Enrichment Evaluation (PROMETHEE) developed by Brans [67] and further extended by Vincke and Brans [139]. PROMETHEE belongs to the family of outranking methods and although not being the most non-compensatory approach. PROMETHEE is a quite simple ranking method in conception and application compared with the other methods for multi-criteria analysis [90], [66]. One of the extensions of PROMETHEE (PROMETHEE II) enables a complete ranking of alternatives, while other approaches provide partial rankings including possible incomparabilities.

PROMETHEE has a widespread use in decision-making situations varying from environmental management to business and financial management, medical applications, etc. A comprehensive review of PROMETHEE methodologies and applications is provided in [61].

6.4.2 *PROMETHEE*

In order to use PROMETHEE it is necessary to provide additional information between the criteria and within each criterion, as described in the following paragraphs.

Three main criteria were considered: environmental, economic and user costs. The environmental criteria considered in the analysis included abiotic depletion, acidification, eutrophication, global warming, ozone depletion, human toxicity, ecotoxicity and photo oxidant formation. The economic criteria included construction cost, management costs and end-of-life costs. For user costs a single criterion was considered representing traffic delay costs, vehicle operation costs and accident costs.

Information between criteria is given by a set of weights $\{w_j, j = 1, 2,\ldots, k\}$ representing the relative importance of the different criteria. The higher the weighting factor the more important the criterion. It is up to the user to define the set of weighting factors to be assigned to each criterion.

The information within each criterion, the preference structure, is based on pairwise comparisons. The deviation between the evaluations of two alternatives on a particular criterion is considered. For small deviations, the decision-maker allocates a small preference to the best alternative or possibly no preference if the deviation is negligible. The larger the deviation, the larger the preference.

For the analysis of the case studies presented in the previous sections, different scenarios were considered for the weighting of different criteria:

- **Scenario 1** considered equal importance for the three main criteria: environmental, economical and user costs (1/1/1);
- **Scenario 2** considered a higher importance to the environmental criterion in relation to economical and user costs (2/1/1);
- **Scenario 3** considered a higher importance to the economical criterion in relation to environmental and user costs (1/2/1);
- **Scenario 4** considered a higher importance to the user costs in relation to environmental and economical criteria (1/1/2).

The adopted approach was applied to the case studies performed in this project and the results are provided in the following sections. In all cases, the “day work” scenario was considered for the comparison of results.

6.4.3 *Case studies A*

The combination of different criteria is illustrated in Figure 6-22 for the different weighting scenarios described in the previous section. Higher values correspond to higher rakings and thus, better performance.

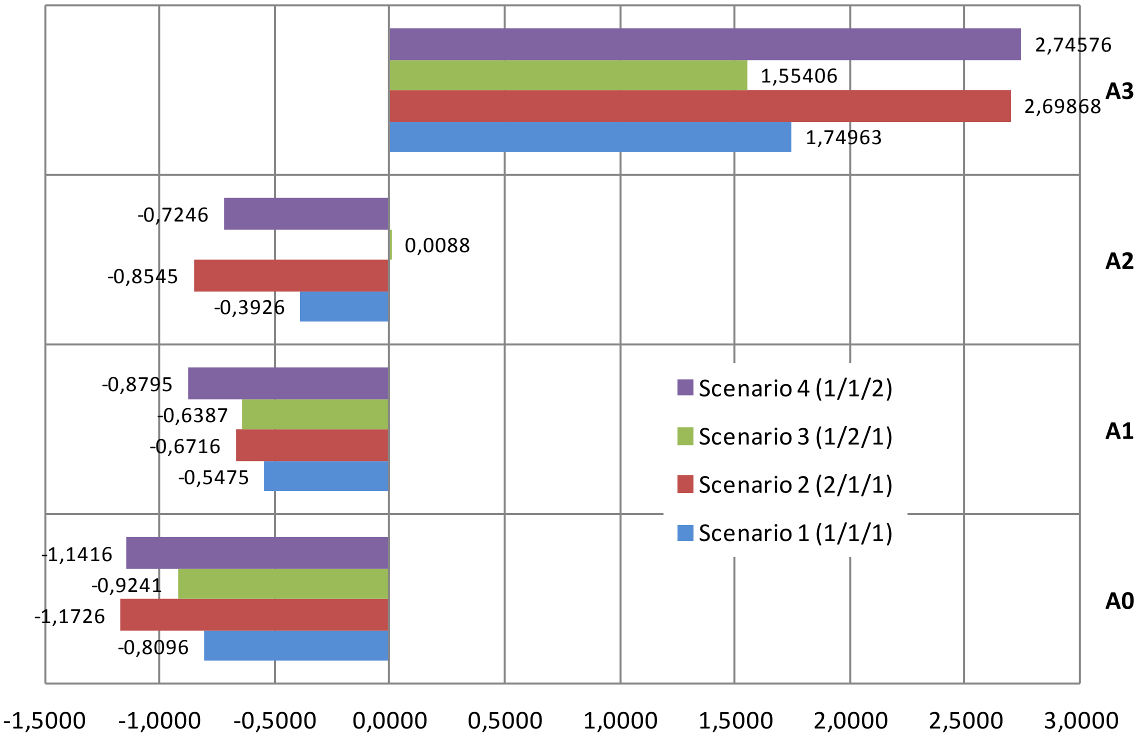


Figure 6-22. Multi-criteria analysis for case studies A1. A2. A3 and A4.

As observed from the previous figure, case study A3, by providing three lanes in each direction, has always a better ranking, independently of the weighting scenario. This result was already expected from the analysis of the different criteria presented in the previous sub-chapters of this report. Traffic congestion due to work activity is a major cause of environmental impacts and user costs. Consequently, minimizing this problem leads to a better life-cycle performance.

6.4.4 *Case studies B*

Case studies B0-1, B0-2, B0-3, B0-4, B0-5, B1-1, B1-2, B2-1 and B2-2 provide the same functional unit. However, due to different geometric characteristics of the deck, for the comparison of results and for the application of the multi-criteria approach, the results of the case studies were normalized by the area of each deck. Therefore, the result of each criterion was considered in unit/m^2.

In addition, for the purpose of the application of the multi-criteria approach, the results of case studies B2-1 and B2-2 were simplified in order to fulfil the same system boundary of the remaining case studies.

Therefore, the combination of different criteria is illustrated in Figure 6-23 for the different weighting scenarios described in the previous section. As already referred, higher value corresponds to higher ranking.

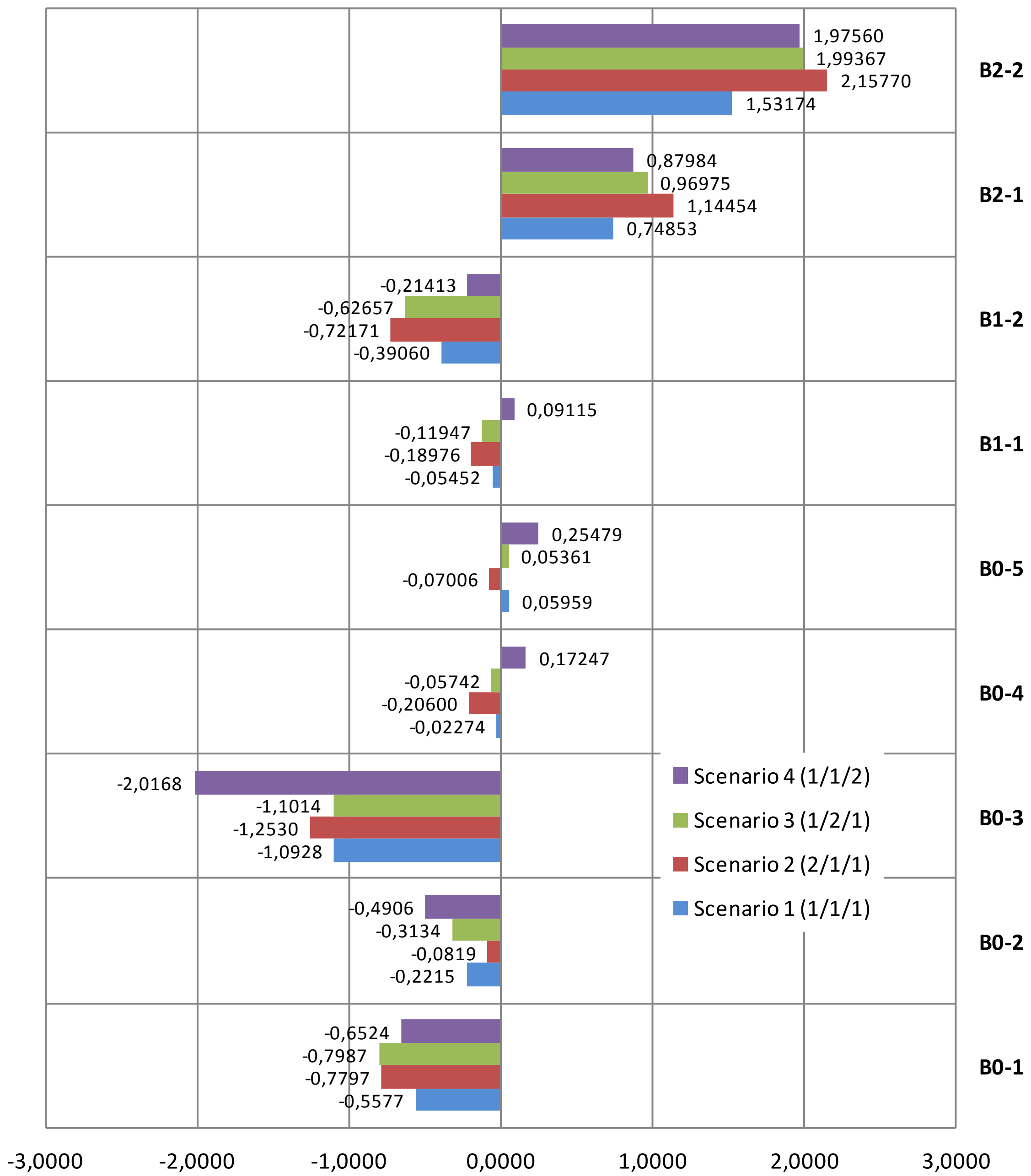


Figure 6-23. Multi-criteria analysis for case studies B0-1, B0-2, B0-3, B0-4, B0-5, B1-1, B1-2, B2-1 and B2-2.

As observed from the previous Figure 6-23, case studies B2-1 and B2-2 have always a better ranking in comparison with the remaining case studies, independently of the weighting scenario.

Independently of the weighting scenario the holistic approach reflected by the results of the multi-criteria analysis shows clear advantages for steel-composite bridges in comparison with concrete solutions, see results of B0-2 and B0-3. However one should keep in mind from the other more detailed comparisons that this is mainly due to the consequent consideration of the traffic interference with the motorway which is crossed by bridge case B.

To conclude, it appears that for such short spans, integral abutments should always be preferred to usual abutments (with bearings and expansion joints). Then the choice between a concrete bridge and a steel-concrete composite bridge is partially governed by the importance given to user costs and therefore depend on the position of the bridge in the transport network.

6.4.5 *Case studies C*

For case studies C0, C1-1 and C1-2, the analysis of different criteria has led to the results illustrated in Figure 6-24 for the different weighting scenarios. Once again, it is noted that higher value corresponds to higher raking.

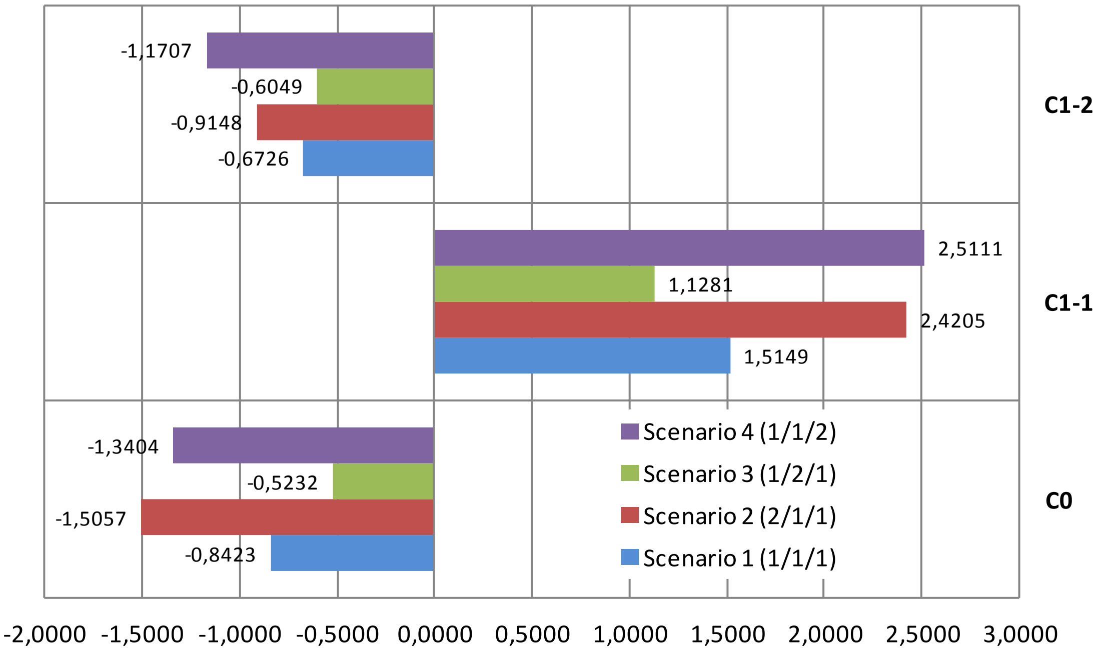


Figure 6-24. Multi-criteria analysis for case studies C0. C1-1 and C1-2.

As observed from the previous figure, case study C1-1 has always a better ranking, independently of the weighting scenario. Likewise, this result was already expected from the analysis of the different criteria presented in the previous sub-chapters 6.3 of this report. Being a two-deck bridge it is possible to divert the traffic during maintenance actions from one deck to the other, enabling two lanes of traffic in each direction. Hence, the impacts due to traffic are that crucial to influence the overall performance. Once the traffic congestion is reduced the overall results are improved.

7 HANDBOOK AND SBRI-TOOL

7.1 General

Besides this final report the essential research results were prepared in a *handbook*, allowing for practical application.
The developed methodology is implemented in a user-friendly software *tool* which enables the calculation of LCA and LCC for bridges, and the comparison of alternative solutions by means of a multi-criteria decision analysis.
Both, the handbook and the SBRI-tool are available for free under the following address of the project partner ArcelorMittal:

http://www.arcelormittal.com/sections/en/library/steel-research-reports/bridges.html

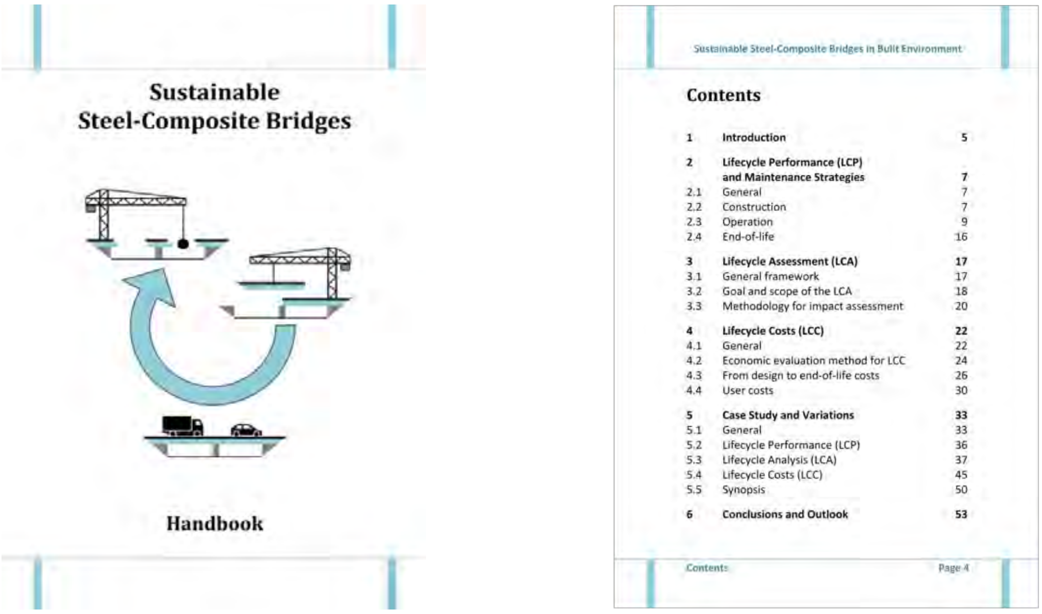
or from the website of the European Convention for Constructional Steelwork (ECCS):

www.steelconstruct.com (under Projects>Member projects>SBRI>Free Tools)

Any update, if needed, will be communicated under the same addresses.

7.2 Handbook – Sustainable Steel-Composite Bridges

The objective of the handbook [40] is to provide an overview about the most important relationships and to describe the complete process in essence in order to make the research results accessible for engineers, designers and stakeholders. The life-cycle analysis is described in a concise way with the necessary information about the life-cycle performance and maintenance strategies as well as about the approach for life-cycle assessment and life-cycle costs. The complete process is demonstrated by a case study covering the entire lifespan with the comparison of three different types of bridges crossing a motorway. As result the integral the steel-composite bridge shows the best performance in regard of sustainability.
The handbook comprises 60 pages and is illustrated by 19 tables and 22 figures, see Figure 7-1.



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2	Lifecycle Performance (LCP) and Maintenance Strategies	7
2.1	General	7
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Figure 7-1. SBRI-Handbook.

7.3 SBRI-tool

The software tool is developed in the software FileMaker® as a modular program. The main body of the program, which is illustrated in Figure 7-2, is used for processing of data and quantification of results. In addition the program incorporates the databank developed in relation to environmental and costs data of materials and processes, and a module for decision making.

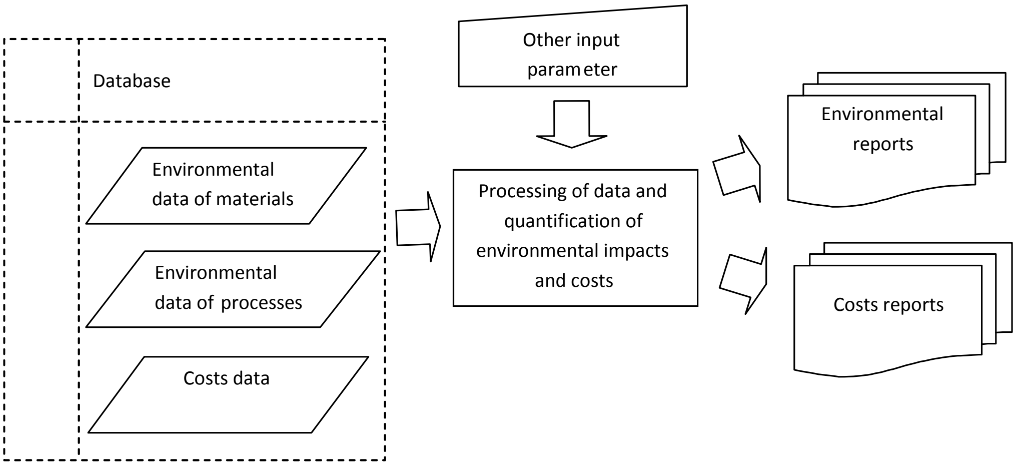


Figure 7-2. General framework of the software.

The main interface of the program is illustrated in Figure 7-3. The program was calibrated by the case studies performed throughout the project. The program is organized in three main modules: (i) databases, (ii) lifetime analyses and (iii) multi-criteria analysis.

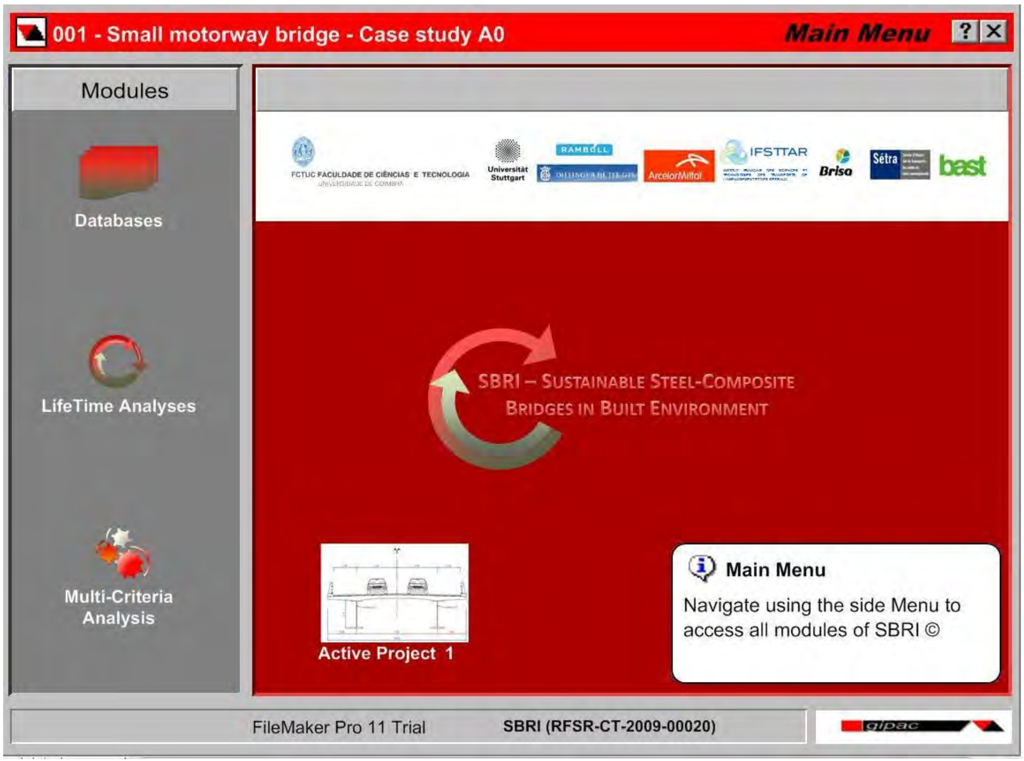


Figure 7-3. Main menu of the software.

The module databases enable the input of all the information from each project. In addition, the most relevant plans of the projects can also be introduced in order to enable a better understanding of the project. Then, for each project, it is necessary to introduce the number of stages and the content of each stage. For each main stage (construction, operation and end-of-life) a template is provided, and the user needs only to fill the relevant fields.

The introduction of data is given in Figure 7-4 and the output of the software in Figure 7-5.

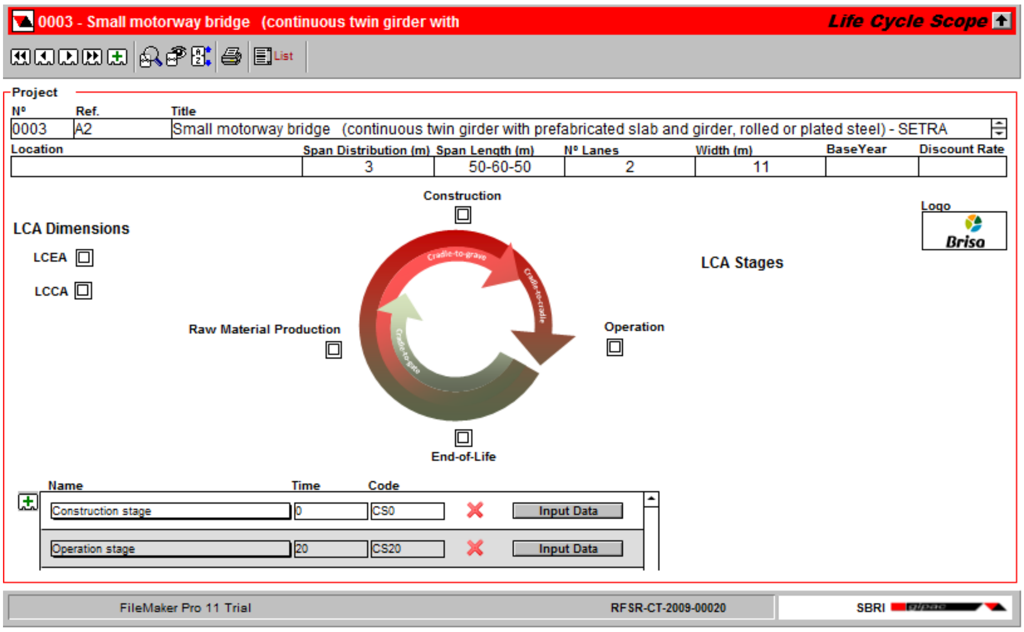
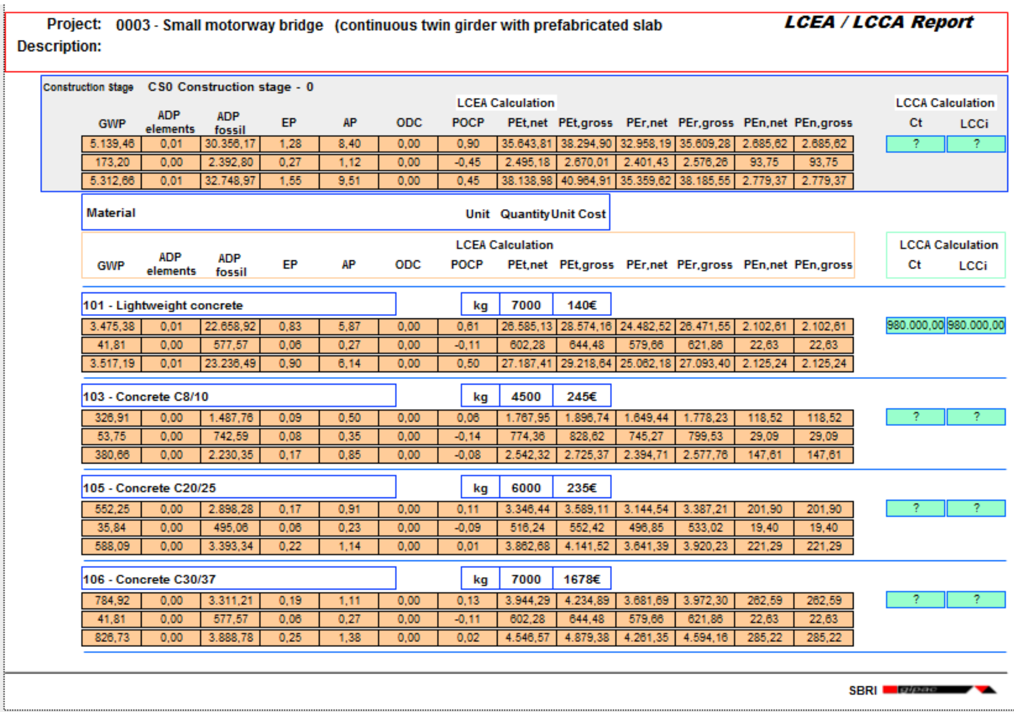


Figure 7-4. Introduction of input data.

When all the relevant information is introduced, the life-cycle environmental and costs analyses are performed and different output reports are created. The user may select between a detailed report for each stage or a summary report.



Project: 0003 - Small motorway bridge (continuous twin girder with prefabricated slab — ***LCEA / LCCA Report***

Description:

Construction stage — CS0 Construction stage - 0

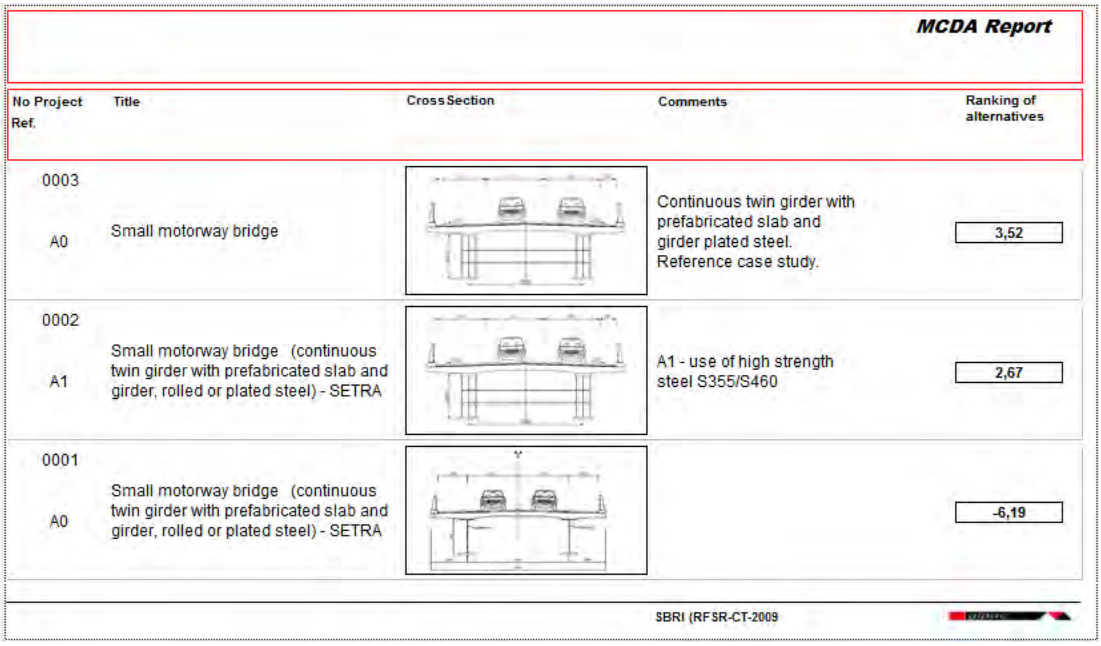
	GWP	ADP elements	ADP fossil	EP	AP	ODC	POCP	PEt,net	PEt,gross	PEr,net	PEr,gross	PEn,net	PEn,gross	Ct	LCCi
	5.139,46	0,01	30.356,17	1,28	8,40	0,00	0,90	35.643,81	38.294,90	32.958,19	35.609,28	2.685,62	2.685,62	?	?
	173,20	0,00	2.392,80	0,27	1,12	0,00	-0,45	2.495,18	2.670,01	2.401,43	2.576,26	93,75	93,75		
	5.312,66	0,01	32.748,97	1,55	9,51	0,00	0,45	38.138,98	40.964,91	35.359,62	38.185,55	2.779,37	2.779,37		

Material	Unit	Quantity	Unit Cost
101 - Lightweight concrete	kg	7000	140€
103 - Concrete C8/10	kg	4500	245€
105 - Concrete C20/25	kg	6000	235€
106 - Concrete C30/37	kg	7000	1678€

Material	GWP	ADP elements	ADP fossil	EP	AP	ODC	POCP	PEt,net	PEt,gross	PEr,net	PEr,gross	PEn,net	PEn,gross	Ct	LCCi
101 - Lightweight concrete	3.475,38	0,01	22.658,92	0,83	5,87	0,00	0,61	26.585,13	28.574,16	24.482,52	26.471,55	2.102,61	2.102,61	980.000,00	980.000,00
	41,81	0,00	577,57	0,06	0,27	0,00	-0,11	602,28	644,48	579,66	621,86	22,63	22,63		
	3.517,19	0,01	23.236,49	0,90	6,14	0,00	0,50	27.187,41	29.218,64	25.062,18	27.093,40	2.125,24	2.125,24		
103 - Concrete C8/10	326,91	0,00	1.487,76	0,09	0,50	0,00	0,06	1.767,95	1.896,74	1.649,44	1.778,23	118,52	118,52	?	?
	53,75	0,00	742,59	0,08	0,35	0,00	-0,14	774,36	828,62	745,27	799,53	29,09	29,09		
	380,66	0,00	2.230,35	0,17	0,85	0,00	-0,08	2.542,32	2.725,37	2.394,71	2.577,76	147,61	147,61		
105 - Concrete C20/25	552,25	0,00	2.898,28	0,17	0,91	0,00	0,11	3.346,44	3.589,11	3.144,54	3.387,21	201,90	201,90	?	?
	35,84	0,00	495,06	0,06	0,23	0,00	-0,09	516,24	552,42	496,85	533,02	19,40	19,40		
	588,09	0,00	3.393,34	0,22	1,14	0,00	0,01	3.862,68	4.141,52	3.641,39	3.920,23	221,29	221,29		
106 - Concrete C30/37	784,92	0,00	3.311,21	0,19	1,11	0,00	0,13	3.944,29	4.234,89	3.681,69	3.972,30	262,59	262,59	?	?
	41,81	0,00	577,57	0,06	0,27	0,00	-0,11	602,28	644,48	579,66	621,86	22,63	22,63		
	826,73	0,00	3.888,78	0,25	1,38	0,00	0,02	4.546,57	4.879,38	4.261,35	4.594,16	285,22	285,22		

SBRI

Figure 7-5. Output of software.



MCDA Report

No Project Ref.	Title	Cross Section	Comments	Ranking of alternatives
0003 A0	Small motorway bridge		Continuous twin girder with prefabricated slab and girder plated steel. Reference case study.	3,52
0002 A1	Small motorway bridge (continuous twin girder with prefabricated slab and girder, rolled or plated steel) - SETRA		A1 - use of high strength steel S355/S460	2,67
0001 A0	Small motorway bridge (continuous twin girder with prefabricated slab and girder, rolled or plated steel) - SETRA			-6,19

SBRI (RFSR-CT-2009

Figure 7-6. Ranking of alternative bridge solutions.

Furthermore, the software enables the comparison and ranking of alternative solutions, through the implementation of the multi-criteria decision approach described in section 6.4. The output of the multi-criteria analysis is a list of the alternatives by decreased order of importance, as illustrated in Figure 7-6.

8 SUMMARY AND OUTLOOK

8.1 Summary

In this project European partners from universities, research centres, road administrations, design offices and steel producers brought together their knowledge and experiences on steel-composite bridges. These bridges were not as commonly regarded only under the aspect of an efficient initial state and construction cost performance but over the entire life-cycle. On the one hand, during this long life-cycle bridges are designed for, degradation processes such as fatigue, corrosion and carbonation were regarded. On the other side inspection and maintenance were closely looked at in order to keep the bridges in good conditions. The functional quality was combined with the environmental and economic quality. By this holistic approach (LCP, LCA and LCC) an assessment over the life-cycle was reached.

Typical for the application range of steel and steel-composite bridges in Europe three bridge types were identified according to the span length and their functionality. Small motorway bridges spanning around 50 to 60 meters using composite girders were considered as bridges of type A. Bridges of type B consisted of crossings of motorways and allowed for comparisons between different span distributions (integral, two-span and three-span bridges). Span lengths up to 120 meters were reached by big motorway bridges and were assigned to type C with box-girder composite sections.

As given in Table 2-1 an extensive number of case studies was selected, assigned to the bridge types and studied in detail. The base of the analyses was the extensive data collection performed and compared for the different European countries involved. The detailed data compilation was integrated into a database. Bridges are designed for more than 100 years and the operation stage plays a central role. Special focus was given by compiling and comparing inspection and maintenance strategies. Standard scenarios were elaborated including frequencies, costs, traffic restrictions during the actions, equipment etc. For the maintenance scenarios average service lives of bridge elements were defined and necessary actions described. In addition two specific scenarios were elaborated and studied: a) the lack of money scenario and b) the prolonged life scenario. For both of these scenarios further assumptions were made and an adapted frequency of maintenance actions assumed.

The case studies were designed following the requirements of codes and rules. The material quantities determined were further used in life-cycle costs and life-cycle environmental analyses. Bridges immediately start to deteriorate after entering into service. In order to keep the bridges above a required condition, inspection, maintenance and rehabilitation actions are necessary. With each intervention environmental and economic impacts are caused and need to be taken into account. Hence, the structural performance must be known as environmental and economic analysis directly depend on it.

Degradation can be divided into several processes; here the focus was put on detailed investigations on fatigue, corrosion and carbonation. During four fatigue tests on a standard detail of steel bridge girders, the transverse stiffener, non-destructive test methods for early crack detection were compared. The innovative method of Phased Array made crack detection possible for cracks starting from the weld toe at an earlier stage than the commonly used methods of magnetic particle or penetration testing. Some limitations of applications were experienced for their use on bridge coating. An increase of the fatigue resistance was reached by the effective post-weld treatment method of high frequent hammering applied to the girders. This improved fatigue resistance was integrated in the design of a case study. The influence of cracked concrete to the behaviour of horizontally lying shear studs was tested in 3 static and 7 fatigue tests. A significant decrease due to the cracked concrete in comparison to former tests was noticed.

In corrosion tests facing the common problem of the joint between steel beam and the concrete deck five different types of coatings were tested. As expected, the most complete coating system (Type 5) led to the best performance. However no major difference has been observed between the Type 5 and Type 4 coating system, which corresponds to standard coating systems of motorway bridges. Considering an environmental category with medium corrosivity (C3) the service life of the coating systems Type 4 and 5 is estimated to be more than 20 years, which provides useful information for the prediction of the maintenance plan of composite bridges.

As a guide for the choice of the maintenance timing for carbonation the Bakker model was studied in the probabilistic framework. An optimisation function was defined in terms of costs and safety.

For the life-cycle environmental analysis a system boundary was set, including all stages over the complete life-cycle of the bridges, from raw material extraction to the end-of-life, considering also the recycling. Eight indicators of environmental performance were regarded and the inputs determined.

Life-cycle costs were regarded from design to the end-of-life of the bridges. In order to compare past and future cash-flows with those of today several methods were compared. A yearly discount rate was set to 2% in the LCCA for the 100-year service life. In addition to direct measurable costs user costs were regarded. Those costs are caused by maintenance operations leading to traffic congestion or disruption of the normal traffic flow. User costs were divided into traffic delay costs and vehicle operating costs.

The established database, the analysis of life-cycle performance, the life-cycle environmental and economic analyses were all together integrated in the holistic approach to the aforementioned and selected case studies.

The following conclusions can be provided for each bridge type:

<u>*Case A - Small motorway bridges:*</u>

In this case study for small motorway bridges, it can be observed from the life-cycle environmental analysis that the stages of material production and operation are by far dominating all impact categories. In terms of processes, the production of construction materials throughout the life-cycle and traffic congestion due to work activity, are the main causes of environmental burdens in the life-cycle analysis. For the operation stage the impacts are mainly caused by traffic congestion. The overall results are improved the most by allowing three lanes of traffic as therewith congestion is reduced. The reduction of the structural steel quantity by the use of HSS and its increase by the use of self-weathering steel explains the improvement and the worsening of the results of the LCA respectively. It is observed that due to no traffic under the bridge the use of self-weathering steel has no major advantage.

The construction costs in parallel to the LCA were reduced for the use of HSS but increased for the use of self-weathering steel. Looking at total life-cycle costs the ranking was changed as the most advantageous solution of case A appears to be the application of self-weathering steel. The increase of construction costs for the three-lane section was significantly reduced by the application of post-weld treatment. User costs are reduced for the night work scenario. Both three-lane variants sum up to the least user costs.

A combination of different criteria for this bridge type shows that independently of the weighting scenario, providing of a third lane in each direction leads always to a better ranking. This shows that traffic congestion due to work activity cause major environmental impacts and user costs.

<u>*Case B - Crossings of motorways*</u>

For crossings of motorways the environmental impacts of the material production and the operation stage dominate by far the life-cycle. In general a reduction of these impacts is achieved as well for integral bridge solutions as for three-span bridges. Also the concrete variants B0-2 and B-3 show advantages. Taking the use of equipment into account a significant contribution is noticed to GW, AP and EP, whereas the transportation of materials is negligible. For crossings of a motorway a big benefit was registered for the use of self-weathering steel as the maintenance of any coating is avoided and therewith traffic congestion reduced.

Looking at the construction costs especially for integral solutions the foundations play a decisive role. In total the three-span solutions are more expensive, being relativized if taking the increased bridge length into account. The integral bridges significantly reduce operating costs due to the lack of maintenance actions concerning expansion joints. A similar reduction is noted for the use of self-weathering steel significantly reducing the maintenance costs due to the lack of costly maintenance actions concerning to corrosion protection. The design of integral bridges requires less maintenance and therefore leads to less traffic disruption and reduced user costs. The three-span bridges on the other extreme have large user costs as a higher quantity of elements requires maintenance and therefore traffic disruption is caused.

It is noted that the concrete alternatives B0-2 and B0-3 have lower operation costs than the standard steel-composite solution B0-1 due to the fact that concrete bridges do not require the maintenance actions for corrosion protection. However, they have more concrete surface to maintain which requires traffic disruptions of the highway underneath the bridge and causes higher user costs.

As the variants of this case have different geometric characteristics a comparison was made per square meter of bridge area. Surprisingly the combination of the different criteria observed a best ranking for the three-span bridges. In a traditional point of view, based on the construction costs the concrete bridge cast in place would have been chosen. The ranking of the different bridge variants according to life-cycle costs and according to life-cycle environmental analysis is the same. Once the user costs are taken into account, the ranking is changed in the way that the integral steel-composite bridge is preferred, the usual steel-composite bridge last and the concrete bridge in the middle of both composite bridges.

To conclude, it appears that for such short spans, integral abutments should always be preferred to usual abutments (with bearings and expansion joints). Also the choice between a concrete bridge and a steel-concrete composite bridge is governed by the importance given to user costs and therefore to the position of the bridge in the transport network.

Case C - Big motorway bridges

Similar conclusions on the life-cycle environmental analysis can be drawn for the big motorway bridges, as before again the material production and the operation stage dominate the life-cycle. For the Global warming the input from the traffic congestion is not as big as for the other categories where as for the Photochemical oxidation the production of materials contributes by far less than the traffic congestion. For the two-deck bridge it is possible to significantly reduce traffic congestion as traffic is diverted during maintenance actions. A reasonable improvement of the environmental impacts was achieved by the use of HSS. Contrary to the previous case studies, the "lack of money" scenario slightly increases the results of most impact categories, as major maintenance actions take place in years 70 and 90.

Regarding the life-cycle costs, the variant using HSS allows for the lowest construction costs and therefore for smallest total costs. The two-deck solution on the one hand is the most costly one regarding life-cycle costs but on the other hand reduces user costs significantly.

A holistic approach to the type of big motorway bridges shows that the two-deck bridge comes to the best ranking independently of the weighting scenario. This shows that even the most expensive variant regarding the initial costs, can be the preferred with an integral approach.

It has been recognised that the idea of sustainable design also needs the transfer to practice. In addition to these detailed analyses of case studies a handbook has been prepared, describing the life-cycle performance and maintenance strategies, the life-cycle assessment and the life-cycle costs analysis along the life-cycle of bridges in a concise way. The handbook should enable the direct application of the method by users by providing the necessary information in a condensed form. A case study and its variants are also given in the handbook in order to show an example and facilitate the use in practice.

The applied methodology is implemented in a user-friendly software tool which enables the calculation of case studies and the comparative analysis between alternative solutions.

8.2 Outlook

By this research project a range of standard European steel-composite road bridge situations was covered. Analyses included the situation of small-span-bridge crossing a motorway where typically concrete bridges dominate due to their low initial costs. It could be demonstrated by the case studies that considering sustainability and a life-cycle approach might bring advantages to steel solutions. But this strongly depends on the assumptions taken e.g. the maintenance scenarios play a decisive role because of the significant influence of the operation/maintenance stage. So, more applications and comparisons are needed to identify where possible chances of steel-composite bridges exist.

Valuable knowledge has been gathered in an extensive databank considering especially realistic data for inspection and maintenance. A useful software tool has been developed integrating the databank. And a handbook gives guidance to users on how to apply the methods to real bridge situations. However this knowledge is so far not well spread and therefore only accessible to interested experts.

During the SBRI-Workshop, where a selected number of experts participated, during discussions helpful feedback to the project was gained. Also a large interest was noticed to the new approach of sustainable bridge design by the participating engineers coming from design offices, road authorities and other stakeholders. But still, the method is yet not widely spread and well known. In order to reach an enlarged dissemination of the research results further workshops/seminars should be organized in different countries. This also could be facilitated by translations of the SBRI-Handbook and SBRI-tool. An introduction to the use of the software tool could enable its use in the daily work.

Furthermore by applying the method to built bridge examples such a dissemination project could allow for interesting comparisons and demonstrate the chances and advantages of this new holistic approach of sustainable design for steel-composite bridges. The SBRI-tool and guidance could be further developed and improved and the database extended for conditions in additional countries. Also the number of defined bridge types may be enlarged. This should finally lead to a wider acceptance of a necessary shift from the construction-cost dominated tender stage to a holistic life-cycle design for bridges across Europe.

It should clearly be stated that sustainable design of bridges is different from sustainable design of buildings. Whereas criteria of sustainability for buildings are largely independent from the local surroundings and conditions and are mainly dominated by the questions of energy consumption, sustainable bridge design is strongly dependent on local conditions such as e.g. the traffic situation or climatic conditions. And because of the long operation phase of 100 years, durability and questions of efficient maintenance are decisive. That is also why a ranking system, giving certificates (gold, silver, bronze) to structures, like it is done for buildings, is not suitable for bridges. Bridges need to be treated more individually.

However an accepted evaluation scheme according to criteria of sustainability might allow comparing different bridge solutions for a given situation and by that, give objective criteria to bridge planners and authorities to focus on one solution and neglect another. So it might be worthwhile to invest in an extra lane as option (see case A3 in the case studies) in order to gain the liberty to allocate undisturbed maintenance actions and the possibility of increased traffic in the long run. Or an extra investment in weathering steels might pay off in the whole life-cycle of a bridge because this leads to savings for corrosion protection and its maintenance.

So if the aim of the holistic analysis is to identify the possibilities of improvement of the processes with major contribution to the impacts, then the structural, environmental and cost performances should be interpreted individually by the newly developed method. If the aim of the analysis is to support a decision making problem, the balance between the individual performances may be achieved by a multi-criteria decision analysis as demonstrated for the case studies. It might be emphasized that a life-cycle analysis is not a decision making approach in itself; but it provides valuable information for decision makers in the process of development and decision. So it is of importance that bridge designers and planners as well as bridge authorities gain an easy access to the developed methods of sustainable bridge design.

In the SBRI-project only road bridges were regarded. Railway bridges are dominated by their embedment into a railway network with interdependences. The databank created in this project and the software tool can therefore not directly be used for railway bridges, as different assumptions especially during the operation stage are to be considered. Also railway bridges have different loadings which have a stronger impact in view of fatigue. Design of railway bridges nevertheless also demand a holistic approach during their entire life-cycle and the transfer of methods developed in this project to the design of railway bridges may be of a major interest.

For bridges in general, and railway bridges in particular, durability is decisive. So e.g. the effect of self-weathering steels to avoid additional maintenance actions for corrosion protection was of high interest for the participants of the SBRI-Workshop and might form a future argument for the use of steel in bridges if spread more widely, [86], [130]. However the handling in view of maintenance and inspections still leaves some open questions which aside of the aesthetic look seem to prevent a wider use.

Also methods to improve the fatigue life such as post-weld treatment are reasonable, especially for high strength steels which suffer from fatigue criteria so that their use cannot fully profit from the increased strength. Methods to improve fatigue behaviour are of interest as soon as for bridges durability governs the design. In the frame of this project only a few aspects of increasing and evaluating fatigue could be dealt due to the limited resources and main reference was given to the accepted Eurocode rules. However it is known that fatigue data underlying Eurocode are derived from a large number of very old tests which did not always comply with modern fabrication techniques. A revision of decisive details could largely improve the knowledge and give advantages to modern fabrication.

So, sustainable design of bridges as applied within this project for modern road bridges includes a number of further chances of development in order to improve the market chances of steel-composite bridges.

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ANNEX: INSPECTION AND MAINTENANCE SCENARIOS

Table A 1. Standard Inspection Scenario

Type of inspection	Standard deck surface			Average inspection frequency			Average Cost (€/bridge)			Average Cost (€/m²)			Average occurence during bridge service life (100 years)			Traffic restrictions					
	Area (m²)			Case			Case			Case						A0		B0		C0	
	A0	B0	C0	A0	B0	C0	A0	B0	C0	A0	B0	C0	A0	B0	C0	under	over	under	over	under	over
Routine or annual				annual	annual	annual	450,00	385,00	855,00	0,2557	0,8556	0,0792	100	100	100	NR	NR	NR	NR	NR	NR
Main, principal or periodic special	1.760	450	10.800	6	6	6	6670,00	3900,00	16400,00	3,79	8,67	1,52	17	17	17	NR	SR+LC	SR+LC	SR+LC	NR	SR+LC
Special, extra or exceptional detailed				2 in 100 years			14350,00	10330,00	22755,00	8,15	22,96	2,11	2	2	2	NR	SR+LC	SR+LC	SR+LC	NR	SR+LC

NOTES (Traffic Restrictions):

NR - no restrictions
SR - speed reduction
LC - 1 lane closed

Table A 2. Standard maintenance scenario

Damage	Maintenance Actions	Average service life (years) Cases A0, B0, C0	Maintenance actions periodicity (years) Cases A0, C0	Maintenance actions periodicity (years) Cases B0	units	Average (All) Cost (€/unit) Case A0, B0, c0	Average (All) capacity rate	Average (All) capacity rate (unit)	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100
Steels																											
Steel girder - used up						1200,00	12	t/ month																			
Corrosion (small points/small areas)	partial surface corrosion protection (1)		35	35	m²	55,00	1,15	m²/hour				x							x							x	
Corrosion (complete renewal)	complete renewal corrosion	35			m²	75,00	1,73	m²/hour						x							x						
Concrete																											
concrete slab - used up (A0, B0, C0)	demolition / replacement	100			m³	290,00	0,22	hour/m³																			x
Corrosion of the reinforcement deck plate	partial renewal		25		m²	350,00	0,08	days/m²				x					x					x					x
Concrete edge beam	partial renewal	40			m	100,00	-	-							x								x				
Concrete edge beam	total replacement	40			m	2440,00	-	-							x								x				
Concrete edge beam repairs	partial renewal		40		m	100,00	-	-				x								x							
Expansion joints																											
broken modules (considering a modular joint) B0	total replacement	40			m	1580,00	3,50	m/day							x								x				
broken modules (considering a modular joint) A0,C0	total replacement	40			m	4670,00	4,50	m/day							x								x				
broken concrete header (repair)	total/partial replacement		10	10	m	80,00	0,75	m/hour	x		x		x				x		x		x				x		x
tightening of bolts	total/partial replacement		10	10	m	200,00	0,25	days/m	x		x		x				x		x		x				x		x
Cleaning			10	10		1,50	40,00	m/day/man	x		x		x				x		x		x				x		x
Bearings																											
Elastomeric bearing - used up - A0, B0	total replacement	35			piece	1430,00	2,00	days/piece						x							x						
Elastomeric bearing - used up - C0	total replacement	35			piece	2995,00	2,00	days/piece						x							x						
Elastomeric bearing (repair)	partial replacement				piece	350,00	1,50	days/piece			x							x							x		
Calote bearing - used up - B0	total replacement	100			piece	8000,00	2,00	days/piece																			x
Calote bearing - maintenance	total/partial replacement	100	0	0	piece	0,00	-	-																			x
Corrosion of metalic elements (Sa2/St3)	painting of metalic elements		35	35	piece	465,00	2,50	piece/day						x							x						
Road surface																											
cracks, ruts, excavation	total replacement	20			m²	11,50	1400,00	m²/hour			x				x				x				x				x
cracks, ruts, excavation	minor repairs		10	10	m²	11,40	1400,00	m²/hour	x				x				x				x				x		
Water proofing layer																											
cracks, ruts, excavation	total replacement	40	0		m²	40,00	0,01	h/m²							x								x				
Railings																											
used up	total replacement of railings	40			m	140,00	1,75	m/hour							x								x				
painting	painting of metalic elements		20		m	19,00	2,00	m²/hour			x								x								
Gutters																											
replacement dewatewring	total replacement	25	0		m²	89,00	0,10	days /m				x					x					x					x
Safety barrier																											
used up	total replacement of safety barrier	25	25		m	420,00	4,50	m/hour							x								x				
safety barriers - minor repairs	total/partial replacement				m²	50,00	1,30	m²/hour				x								x							x

Table A 3. Types of materials, units of these materials and traffic restrictions

Damage	Maintenance Actions	Traffic restrictions				Materials	units
		Case A0,C0	Case B0	under	over		
Steels							
Steel girder - used up	demolition / replacement	A0,C0	B0	1lane closed per day/per carriageway	Road closed	S355N/NL	t
Corrosion (small points)	repainting surface corrosion protection		B0	1lane closed per day/per carriageway	no restrictions	C4 ANV	m^2
Corrosion (some areas)	partial renewal corrosion protection		B0	1lane closed per day/per carriageway	no restrictions	C4 ANV	m^2
Corrosion (complete renewall)	complete renewal corrosion protection		B0	1lane closed per day/per carriageway	no restrictions	C4 ANV	m^2
Concrete							
concrete slab - used up	demolition / replacement		B0	1lane closed per day/per carriageway	Road closed	C35/45	m^2
Corrosion of the reinforcement deck plate	partial renewal		B0	1lane closed per day/per carriageway	1 lane closed per day	S355	m^2
Concrete edge beam	replacement	A0,C0	B0	no restrictions	speed reduction		m
Concrete edge beam	partial renewal	A0,C0	B0	no restrictions	speed reduction		m
Concrete edge beam (repairs)	partial renewal	A0,C0	B0	no restrictions	speed reduction		t
Expansion joints							
broken modules (considering a modular joint) B0	total replacement		B0	no restrictions	1 lane closed per day	reinforced elastomeric	m
broken modules (considering a modular joint) A0, C0	total replacement	A0		no restrictions	1 lane closed per day		m
		C0					m
broken concrete header (repair)	total/partial replacement	A0,C0	B0	no restrictions	1 lane closed per day	mortar	m
tightening of bolts	total/partial replacement	A0,C0	B0	no restrictions	1 lane closed per day	-	m
cleaning of debris in the rubber modules (movement restrictions)	Cleaning (blowing and sweaping)	A0,C0	B0	no restrictions	1 lane closed per day	-	m
Bearings							
Elastomeric bearing - used up (B0)	total replacement	A0,C0	B0	no restrictions	speed reduction	high strength steel and elastomer	piece
Elastomeric bearing used up (A0, C0)	total replacement	A0,C0	B0	no restrictions	speed reduction		piece
Elastomeric bearing - repair (B0)	partial replacement		B0	no restrictions	speed reduction		piece
Elastomeric bearing - repair (A0, C0))	partial replacement	A0, C0		no restrictions	speed reduction		piece
calotte bearing - used up	total replacement	A0,C0	B0	no restrictions	speed reduction	steel St 52-3	piece
calotte bearing - maintenance	Cleaning	A0, C0		no restrictions	no restriction	steel St 52-3	piece
corrosion of metalic elements (Sa2/St3)	repainting of metalic elements	A0,C0	B0	no restrictions	speed reduction	steel	piece
Road surface							
cracks, ruts, excavation	total replacement	A0,C0	B0	no restrictions	1 lane closed per day	asphalt	m^2
cracks, ruts, excavation	minor repairs	A0,C0	B0	no restrictions	1 lane closed per day	asphalt	m^2
Water proofing layer							
cracks, ruts, excavation	total replacement	A0,C0	B0	no restrictions	1lane closed per day	modified bitumen membrane	m^2
Railings							
used up	total replacement		B0	no restrictions	no restrictions / speed reduction	galvanized steel	m
used up	total replacement	A0, C0		no restrictions	no restrictions / speed reduction	galvanized steel	m
Painting	repainting of metalic elements	A0,C0	B0	no restrictions	no restrictions / speed reduction	galvanized steel	m^2
damage caused by accident	partial replacement	A0,C0	B0	no restrictions	no restrictions / speed reduction	galvanized steel	m
Gutters							
replacement dewatewring	total replacement	A0,C0	B0	no restrictions	no restrictions / speed reduction	aluminium alloy	m^2
Safety barrier							
used up (concrete - 3m)	total replacement	A0,C0	B0	no restrictions	1 lane closed per day	C35/45	m
minor repairs (concrete)	partial replacement	A0,C0	B0	no restrictions	1 lane closed per day	C35/45	m^2
used up (steel safety barrier)	partial replacement	A0,C0	B0	no restrictions	1 lane closed per day	C35/45	m^2
damage caused by accident (galvanized steel)	partial replacement	A0,C0	B0	no restrictions	1 lane closed per day	S235JR	m

Table A 4. Lack of money inspection scenario

Type of inspection	Standard deck surface			Average inspection frequency			Average Cost (€/bridge)			Average Cost (€/m²)			Average occurence during bridge service life (100 years)			Traffic restrictions					
	Area (m²)			Case			Case			Case						A0		B0		C0	
	A0	B0	C0	A0	B0	C0	A0	B0	C0	A0	B0	C0	A0	B0	C0	under	over	under	over	under	over
Routine or annual	1.760	450	10.800	annual	annual	annual	450,00	385,00	855,00	0,2557	0,8556	0,0792	100	100	100	NR	NR	NR	NR	NR	NR
Main, principal or periodic special				5	5	5	7240,00	4600,00	20350,00	7,42	8,80	3,35	20	20	20	NR	SR+LC	SR+LC	SR+LC	NR	SR+LC
Special, extra or exceptional detailed				4 in 100 years			14400,00	10450,00	25500,00	8,18	23,22	2,36	4	4	4	NR	SR+LC	SR+LC	SR+LC	NR	SR+LC

LEGEND (Traffic Restrictions):

NR - no restrictions
SR - speed reduction
LC - 1 lane closed

Table A 5. Lack of money maintenance scenario

Damage	Maintenance Actions	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100
Steels																				
Steel girder - used up	demolition / replacement																			
Corrosion (small points/small areas)	partial surface corrosion protection (1)				x		x								x		x			
Corrosion (complete renewal)	complete renewal corrosion									x										
Concrete																				
concrete slab - used up (A0, B0, C0)	demolition / replacement																			
Corrosion of the reinforcement deck plate	partial renewal									x										
Concrete edge beam	total replacement									x										
Concrete edge beam	total surface treatment						x										x			
Concrete edge beam	partial renewal of surface treatment								x										x	
Concrete edge beam repairs	partial renewal				x										x					
Expansion joints																				
broken modules (considering a modular joint) B0	total replacement									x										
broken modules (considering a modular joint) A0,C0	total replacement									x										
broken concrete header (repair)	total/partial replacement	x		x		x		x				x		x		x		x		
tightening of bolts/ partial module replacement	total/partial replacement	x		x		x		x				x		x		x		x		
Cleaning		x		x		x		x				x		x		x		x		
Bearings																				
Elastomeric bearing - used up - A0, B0	total replacement						x							x						
Elastomeric bearing - used up - C0	total replacement						x							x						
Elastomeric bearing (repair)	partial replacement			x							x							x		
Calote bearing - used up - B0	total replacement																			
Calote bearing - maintenance	total/partial replacement																			
Corrosion of metalic elements (Sa2/St3)	painting of metalic elements						x							x						
Road surface																				
cracks, ruts, excavation	total replacement			x										x						
cracks, ruts, excavation	total survival road surface layer							x										x		
cracks, ruts, excavation	minor repairs	x				x						x				x				
Water proofing layer																				
cracks, ruts, excavation	total replacement									x										
Railings (only for case B.03 and B2)																				
used up	total replacement of railings									x										
painting	painting of metalic elements			x				x						x				x		
damage caused by corrosion	partial replacement							x										x		
Gutters																				
replacement dewatewring	total replacement				x					x					x					
Safety barrier																				
used up (steel)	total replacement of safety barrier									x										
safety barriers - minor repairs due to corrosion	total/partial replacement				x			x							x			x		
damage caused by accident (steel)	partial replacement																			

Table A 6. Prolonged life inspection scenario

Type of inspection	Standard deck surface			Average inspection frequency			Average Cost (€/bridge)			Average Cost (€/m²)			Average occurence during bridge service life (130 years)			Traffic restrictions					
	Area (m²)			Case			Case			Case						A0		B0		C0	
	A0	B0	C0	A0	B0	C0	A0	B0	C0	A0	B0	C0	A0	B0	C0	under	over	under	over	under	over
Routine or annual				annual	annual	annual	450,00	385,00	855,00	0,2557	0,8556	0,0792	130	130	130	NR	NR	NR	NR	NR	NR
Main, principal or periodic special	1.760	450	10.800	6	6	6	7240,00	4600,00	20350,00	7,42	8,80	3,35	22	22	22	NR	SR+LC	SR+LC	SR+LC	NR	SR+LC
Special, extra or exceptional detailed					4 in 130 years		14400,00	10450,00	25500,00	8,18	23,22	2,36	4	4	4	NR	SR+LC	SR+LC	SR+LC	NR	SR+LC

LEGEND (Traffic Restrictions):

NR - no restrictions
SR - speed reduction
LC - 1 lane closed

Table A 7. Prolonged life maintenance scenario

Damage	Maintenance Actions	Years																								
		10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100	105	110	115	120	125	130
Steels																										
Steel girder - used up	demolition / replacement																									
Corrosion (small points/small areas)	partial surface corrosion protection (1)				x							x						x						x		
Corrosion (complete renewal)	complete renewal corrosion protection(1)						x							x						x						
Concrete																										
concrete slab - used up (A0, B0, C0)	demolition / replacement																									
Corrosion of the reinforcement deck plate	partial renewal				x					x					x					x			x		x	
Concrete edge beam	total surface treatment																			x						
Concrete edge beam	partial renewal of surface treatment																						x			
Concrete edge beam	total replacement							x								x										
Concrete edge beam repairs	partial renewal				x								x							x			x		x	
Expansion joints																										
broken modules (considering a modular joint) B0	total replacement							x								x										
broken modules (considering a modular joint) A0,C0	total replacement							x								x										
broken concrete header (repair)	total/partial replacement	x		x		x				x		x		x				x		x		x		x		
tightening of bolts/ partial module replacement	total/partial replacement	x		x		x				x		x		x				x		x		x		x		
Cleaning		x		x		x				x		x		x				x		x		x		x		
Bearings																										
Elastomeric bearing - used up - A0, B0	total replacement						x							x							x					
Elastomeric bearing - used up - C0	total replacement						x							x							x					
Elastomeric bearing (repair)	partial replacement			x							x							x						x		
Calote bearing - used up - B0	total replacement																									
Calote bearing - maintenance	total/partial replacement																									
Corrosion of metalic elements (Sa2/St3)	painting of metalic elements						x							x							x					
Road surface																										
cracks, ruts, excavation	total replacement			x				x				x				x				x						
cracks, ruts, excavation	total survival road surface layer*																							x		
cracks, ruts, excavation	minor repairs	x				x				x				x				x				x				
Water proofing layer																										
cracks, ruts, excavation	total replacement							x								x										
Railings																										
used up	total replacement of railings							x								x										
painting	painting of metalic elements			x								x								x			x			
damage caused by corrosion	partial replacement																						x			
Gutters																										
replacement dewatewring	total replacement				x					x					x					x						
Safety barrier																										
used up	total replacement of safety barrier							x								x										
safety barriers - minor repairs due to corrosion	total/partial replacement				x								x							x			x			
damage caused by accident (steel)	partial replacement																									

* dense layer of asphalt containing a large amount of bitumen, that is placed on top of the existing damaged surface layer (and waterproofing layer).

European Commission

EUR 26322 — Sustainable steel-composite bridges in built environment (SBRI)

Luxembourg: Publications Office of the European Union

2013 — 119 pp. — 21 × 29.7 cm

ISBN 978-92-79-34586-9
doi:10.2777/50286

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KI-NA-26322-EN-N

In this research project a holistic approach has been applied to steel-composite bridges by combining analyses of Lifecycle Assessment (LCA), Lifecycle Costs (LCC) and Lifecycle Performance (LCP). Under the perspective of sustainability an entire lifespan, from the construction to the demolition of a bridge, is regarded. A valuable collection of data on LCA, LCC and LCP is obtained and life cycle scenarios are described including maintenance strategies. By postponing and pre-drawing of measurements optimisation towards cost-effectiveness and low environmental impact is achieved. Three representative types of road bridges were chosen and analysed. A differentiation between small motorway bridges, crossings of motorways and big motorway bridges is made. Complete case studies were performed throughout all aspects. In a second step variations and optimisation were applied. The behaviour of each component of bridges is affected by degradation processes such as fatigue, corrosion and carbonation. To capture the process of degradation is the basis of the description of the life cycle of each specific detail. In own tests for fatigue details (transverse stiffener and horizontally lying shear studs) and corrosion degradation models have been updated. In order to allow for comparisons and elaboration of advantages a multitude of variants were studied including also typical concrete solutions to identify the chances and advantages of steel-composite bridges. A workshop of experts was organised in Paris and gave a positive feedback to the project. Finally a user-friendly software tool and a handbook showing the methods' application to users have been prepared.

Studies and reports

ISBN 978-92-79-34586-9

Publications Office

doi:10.2777/50286