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Master Thesis

# Rehabilitation of a post-tension concrete highway bridge using UHPFRC

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unter der Leitung von

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# Preamble

The thesis was conducted in collaboration between the Vienna University of Technology (Technische Universität Wien) and the EPFL (École Polytechnique Fédérale de Lausanne) on the example of a bridge situated in Germany.

The results of this master thesis are the result of a student's work and the data was adapted to the knowledge of the student. Therefore, these results cannot be directly used on the work in question.

If not stated otherwise all images are drawn by the author.

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# Summary

The rehabilitation of bridges is today, and will increasingly become in the future, a complex problem to tackle as its various aspects, from the necessity of preservation to the evaluation of intervention concepts, have to be considered to achieve a holistic evaluation. The different aspects of the rehabilitation of bridges were regarded on a specific example, the bridge by Taubenstein in Germany, which is a typical German bridge from the perspective of the used materials and cross-sections, the length and the age of the structure. In the first part, the valuation of bridges was discussed regarding their historical and aesthetical values. Additionally, the strategy for the preservation of bridges in Germany and the preservation of structures in general in Switzerland was summarised. To show the difference in approach, one specific aspect, the traffic load computation according to Austrian, German and Swiss guidelines, was compared. In the second part, the examination is performed on the example bridge and is divided into general and a more detailed examination. As groundwork, the resistances and actions were updated, a model with the exact geometry was designed and necessary assumptions were summarised. For the general examination a linear approach was considered whereas for the detailed examination a non-linear behaviour of the structure was studied. The results of the examination show that the structural safety for bending in longitudinal direction was not given. With these findings the intervention concept with the new material UHPFRC was designed in the third part. In order to compare the advantages and disadvantages of this new concept in the sustainability assessment another strengthening concept (with CFRP) was introduced. Moreover, an option of reducing the acting loads instead of increasing the structural strength was discussed. In the last chapter the sustainability assessment was presented considering economical, ecological, social and technical aspects. Where sufficient data is available the two strengthening intervention concepts are compared. It can be shown in this preliminary analysis that UHPFRC has various advantages but further studies would be necessary in order to give quantifiable results for the sustainability assessment of UHPFRC. Lastly, the necessity for a standardised guideline for the maintenance and possible intervention measures of structures as well as for the evaluation of these measures within the European Union was found to be evidently lacking.

# Kurzfassung

Die Instandhaltung von Brückenbauwerken ist bereits heute ein komplexes Problem und wird dies auch in Zukunft verstärkt bleiben. Für eine ganzheitliche Analyse der Problematik müssen verschiedene Punkte, von der Notwendigkeit der Erhaltung bis hin zu der Beurteilung von Interventionskonzepten, berücksichtigt werden. Die verschiedenen Aspekte der Instandhaltung von Brücken werden an dem konkreten Beispiel der Brücke von Taubenstein in Deutschland anaylsiert. Die Brücke von Taubenstein kann aufgrund der verwendenten Materialien und Querschnitte, der Brückenlänge und dem Bauwerksalter, als eine typisch deutsche Brücke gesehen werden. Im ersten Kapitel dieser Diplomarbeit wird die Bewertung von Brücken hinsichlichen historischen und ästethischen Aspekten diskutiert. Zusätzlich werden die Strategien für die Erhaltung von Brücken in Deutschland und die Erhaltung von Bauwerken im Allgemeinen in der Schweiz zusammengefasst. Um die unterschiedlichen Vorgehensweisen zu verdeutlichen, werden die Verkehrslastberechnung nach Deutscher, Österreichischer und Schweizer Norm verglichen. Im zweiten Teil wird die Untersuchung der Beispielbrücke, unterteilt in allgemeine und detaillierte Überprüfung, durchgeführt. Im ersten Schritt werden, als Grundlage für die Überprüfung, unter Anderem die Lasten und Widerstandswerte aktualierst, sowie ein Model mit exakter Brückengeometrie entworfen und die notwendigen Annahmen zusammengefasst. Für die allgemeine Untersuchung wird eine lineare Berechung betrachet, für die detailierrte Überprüfung wird jedoch auch das nichtlineare Verhalten des Tragwerks untersucht. Die Ergebnisse zeigen, dass die Tragfähigkeit der Biegung in Längsrichtung nicht gegeben ist. Im dritten Teil wird mit diesen Resultaten die Verstärkung mit dem neuen Material UHPFRC modelliert. Um die Vor- und Nachteile dieses neuen Konzepts in der Nachhaltigkeitsbewertung vergleichen zu können, wird ein weiteres Konzept (mit CFK) eingeführt. Außerdem wird eine weitere Interventionsmaßnahme – die Verringerung der Lasten anstatt der Erhöhung der Strukturfestigkeit – erörtert. Im letzten Kapitel wird die Methode der Nachhaltigkeitsbewertung, gemessen an wirtschaftlichen, ökologischen, sozialen und technischen Aspekten, vorgestellt. Sofern ausreichende Daten zur Verfügung stehen, werden in einem weiteren Schritt die beiden Verstärkungskonzepte miteinander verglichen. Es konnte bereits in dieser vorläufigen Analyse gezeigt werden, dass die Verwendung von UHPFRC vielfältige Vorteile aufweist, jedoch sind weitere Untersuchungen notwenig, um die Nachhaltigkeitsbewertung von UHPFRC zu quantifizieren. Darüber hinaus wird die Notwendigkeit einer standardisierten Richtlinie für die Instandhaltung und Interventionsmaßnahmen von Bauwerken, sowie für die Beurteilung dieser Maßnahmen innerhalb der Europäischen Union, deutlich.

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# Acronyms

| | |
|---|---|
| AP | Acid Potential |
| ASFINAG | Autobahnen- und Schnellstraßenfinanzierungaktiengesellschaft |
| BK 60 | Brückenklasse 60 |
| BMVBS | Bundesministerium für Verkehr und digitale Infrastruktur |
| CFRP | Carbon-fibre-reinforced polymer |
| DACH | Acronym for the Germany, Austria and Switzerland |
| DIN | Deutsches Insitut für Normung |
| EC | Eurocode |
| EC | Eurocode |
| EN | Europäische Norm (Eurocode) |
| EP | Eutrophication Potential |
| GWP | Global Warming Potential |
| ISO | International Organisation for Standardisation |
| LC | Load Case Combination |
| LCA | Life Cycle Assessment |
| LCC | Life Cycle Costs |
| LCI | Life Cycle Inventory |
| LCIA | Life Cycle Impact Assessment |
| LM | Load Model |
| LM1 | Load Model 1 |
| ODP | Ozone Depletion Potential |
| POCP | Photochemical Ozone Creation Potential |
| R-UHPFRC | Reinforced ultra-high Performance Fibre Reinforced cement-based Composites |
| SIA | Schweizerischer Ingenieur- und Architektenverein |
| TS | Tandem System |
| UDL | Uniform distributed loads |
| UHPFRC | Ultra-high Performance Fibre Reinforced cement-based Composites |

# 1 Introduction

The aim of this thesis is to give a guide on handling existing bridges including the determination of the value of a bridge, the examination, the intervention concept with the new material Ultra-high Performance Fibre Reinforced cement-based Composites (UHPFRC) and its evaluation with a sustainability assessment. All this is presented on the basis of specific example – a prestressed concrete highway bridge near Taubenstein in Germany, hereafter referred to as bridge by Taubenstein. The introduction is divided into the motivation and the structure of the thesis.

## 1.1 Motivation of the thesis

The rehabilitation of existing infrastructure is already today and will in the future increasingly become a complex problem to tackle. All developed countries have invested heavily in infrastructure projects with prestressed concrete structures and these structures have to be maintained. In addition to mechanical influences – for which the buildings were designed – some structures, such as highway bridges, have to resist constant chemical effects. An example of such an aggressive, chemical substance is road salt dissolved in liquid water. This exposition of the concrete leads to a deterioration of the supporting structure and additionally to a reduction of the performance reliability. The premature deterioration causes high maintenance costs and repeated prolonged periods of traffic disruptions, in which the bridge has to be maintained.

The importance of rehabilitation of infrastructure projects will be shown on the example of Austria and Germany. However similar numbers can be assumed for all developed countries. Generally the condition of the infrastructure system plays a decisive role for any country's economy. As fewer and fewer financial resources for development and maintenance of infrastructure are available (Kleister et al. 2013), innovative materials and construction methods in this area are undoubtedly trendsetting. The company (ASFINAG) in charge of Austrian highways alone has 5200 bridges in its infrastructure network (Asfinag 2013) and the Germany federal highway system includes 39106 bridges (BMVBS 2013), which all have to be serviced. On average, the annual maintenance costs of bridges are expected to represent between 1.0-1.5% of the manufacturing cost (Jodl & Jurecka 2007). Hence, with the normal planned service life of bridges of 100 years (DIN EN 1990 2010) the maintenance costs are at least equal to the construction costs.

Out of all these bridges over 90% in Austria and close to 90% in Germany are concrete bridges. In Germany 69.9% of all bridges in the federal highway system are prestressed concrete structures. This shows the relevance of the chosen example the highway bridge by Taubenstein. The prestressed concrete bridge build in the 1960s represents a typical bridge from the perspective of the used material, the bridge length and the age of the structure. Mainly in Germany but also in Austria bridge construction boomed in the 1960s to 1980s, therefore many bridges have reached half of their service life and are frequently subject to damage (BMVBS 2013; Kleister et al. 2013).

Successful rehabilitation measurements are one of the biggest challenges for engineers today. On the one hand the maintenance costs and works should be kept as low and short as possible, on the other hand the maintenance measures must meet all mechanical requirements. According to current research results UHPFRC promises to be an optimal building material for future maintenance projects. By using UHPFRC structures the maintenance costs may massively decrease. This is mainly due to a higher durability of UHPFRC thus maintenance measures would have to be carried out less frequently. This minimises the maintenance costs and increases the reliability of the road network. Another problem is that the road traffic loads have risen and the bridges are not designed for these higher loads. An efficient and sustainable solution for rehabilitation of bridges is therefore of great importance.

In order to establish the most sustainable solution a holistic evaluation concept is needed. The sustainability assessment (chosen as the evaluation concept) considers not only the economic and environmental aspect but also the social elements. Through this approach it can be ensured that all relevant aspects are considered and problem shifting, from one aspect to another, is thus avoided through a complete and comprehensive analysis.

## 1.2 Structure of the thesis

The thesis is organised into six main chapters, the first of which is the present introductory chapter and the last is a chapter on the conclusions. In the following paragraph, a brief description of the contents of the remaining four chapters are presented. The first main section (2 Structure valuation) discusses the value of a structure. For a basis the principal data of the bridge is given and the historical and aesthetical aspects analysed. Additionally the strategy for rehabilitation in Germany and Switzerland are summarised. In the next part (3 Examination) the examination is conducted on the example bridge to show how to determine where and to what degree rehabilitation is necessary. Chapter 4 (Concepts of Intervention) introduces the new material UHPFRC as a possible intervention concept. Additionally one further strengthening measure with CFRP and an intervention by reducing the loads are analysed. Chapter 5 (Sustainability assessment of the intervention concepts) presents the sustainability assessment and gives an overview of the current guidelines and important aspects when evaluating the concepts of intervention of existing structures.

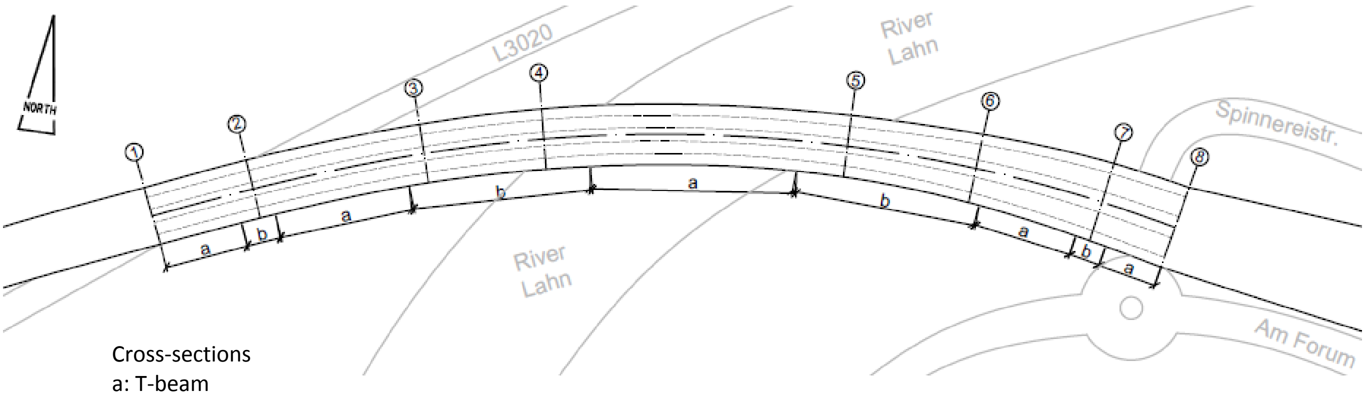
# 2 Structure valuation

The value of a bridge can give an indication on the importance of its preservation. By determining the value of structures – not only from an economic standpoint – it can be establish if and to what magnitude intervention measures are sustainable. For this, various facets should be considered. These can be cultural aspects such as the history and aesthetics of a building, its fulfilment of purpose, the frequency of the use or more material values such as technical, economic and ecological values. In the following two main often neglected aspects – the historical aspect and the aesthetics aspect – will be considered in first instance in general and subsequently for the bridge by Taubenstein. Additionally the strategy for the rehabilitation of existing road bridges of the federal highways in Germany will be discussed. For comparison, the Swiss strategy for the maintenance of existing structures is analysed. The concept is defined mainly with the maintenance value according to SIA 2017. The last section compares the determination of the traffic loads according to the different standards and guidelines of the countries in the DACH region (Germany, Austria and Switzerland) to show on one aspect (technical) the different approaches in this region. As a necessary foundation this chapter begins with the principal data of the bridge by Taubenstein.

## 2.1 Principal Data of the Bridge by Taubenstein

The bridge has a length of about 320 m and a width between 15 m and 28 m, is part of the German federal highway system (highway B49) and was built in 1963. The highway B49 runs in a transitional curve over the river Lahn, the highway L3020 and the Spinnereistraße, as shown in Figure 2.2. The bridge is situated in proximity of the German town Wetzlar. The location of the bridge within Wetzlar and within Germany are shown in Figure 2.1.


*Figure 2.1: Location of the bridge by Taubenstein* (Wikipedia 2016; Google n.d.)




Cross-sections
a: T-beam
b: box girder

*Figure 2.2: Layout plan of the bridge by Taubenstein*

The main structure is a continuous beam over seven fields with eight supporting axes (1-8). The fields have varied spans. The field over the river Lahn has the longest with a span of 93.35 m (Bösche & Curbach 2013a).

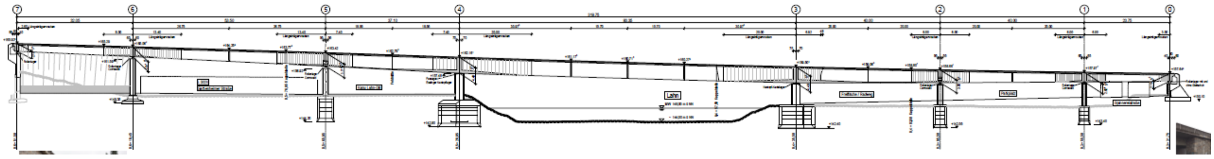


*Figure 2.3: Longitudinal section through the bridge of Taubenstein* (Bösche & Curbach 2013a)

The cross-section of the superstructure is a T-Beam with four webs and is prestressed in the longitudinal and transversal directions, shown on the top Figure 2.4. In the area of the supports the T-beam has an additional compression slab thus the cross-section in these areas is a box girder with three cells, shown on the bottom Figure 2.4. In Figure 2.2 the areas of the T-Beam cross-section are marked with "a" and the areas with the box girder with "b". The total height of the cross-sections varies from 2.1 m to 4.4 m.

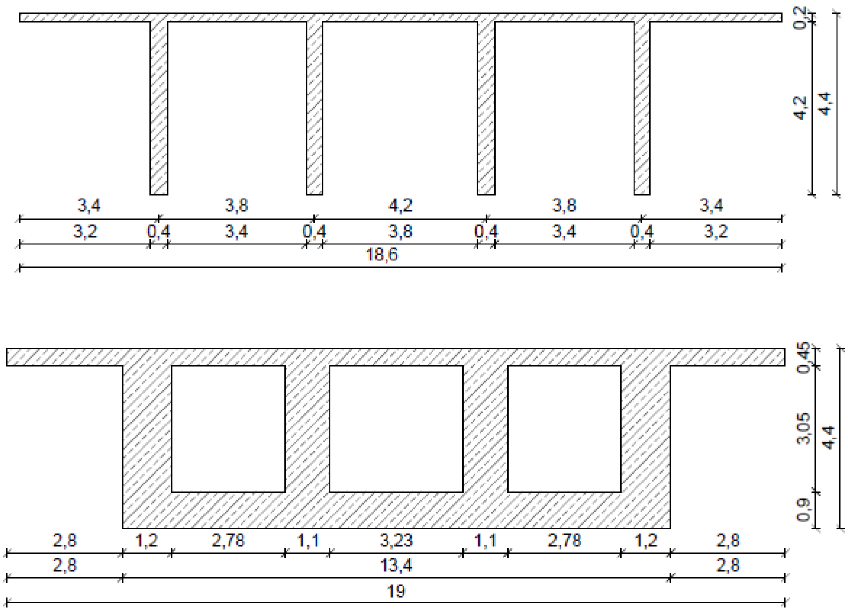



*Figure 2.4: top: cross-section T-beam; bottom: cross-section box girder*

*Figure 2.5: left: bridge over Spinnereistraße-Nordseite; right: bridge over Spinnereistraße-Südseite* (Bösche & Curbach 2013a)

*Figure 2.6: left: bridge over L3020; right: bridge over Lahn* (Bösche & Curbach 2013a)

## 2.2 Historical overview

This short historical overview on bridges focuses on the years between 1945 and 1970 to show the developments which led to the typical bridges of the 60s/70s, during which the highway bridge of Taubenstein was built.

After the Second World War new construction methods and cheaper but stronger building materials were developed. This made the reconstruction possible with the main objective to build fast, cheap and rational (Bühler 2004). The three developments were new construction methods (e.g. classic cantilever method) and the use of new cross-sections (e.g. box girder). However the conception of the new cross-section was only possible due to the invention of prestressed concrete (Pauser 2002). All three innovations are discussed in the following paragraphs.

Since the 1950s the building industry has tried to minimise the use of auxiliary structures, mainly scaffoldings and formwork for the concrete. With the invention of new building methods, for example the classic cantilever construction method (Freivorbau) this goal was achieved. The classic cantilever construction method exploits the state of equilibrium (comparable to the system of a balance beam) in order to enable the construction with little or no scaffolding and with a decreased use of material for concrete beams. The investment for the construction is very low and is generally limited to the traveller (Vorbauwagen). A further advantages of using the cantilever construction method instead of the conventional method is that much larger spans can be built, up to 200 m instead of 70 m. Furthermore the load bearing capacity of the structure can be used immediately (Pauser 2002).

The typical concrete beams, which brought a main step towards modernisation, are for example the box girders (Kastenträger) and the T-Beams (Plattenbalken). A box girder is a beam with a rectangular hollow profile. By saving the material in the core of the profile the weight is minimised whilst having favourable load-bearing and torsion behaviour (Bühler 2004). The precondition for the advancement of these concrete beams was the development of high-quality but nonetheless low-cost pre-stressing or post-stressing tendons and the improvement of tensioning technologies. The first step towards the development of prestressed concrete was the understanding of the long-term deformation of concrete due to compression - the creep deformation. E. Freyssinet recognised in 1928 that in order to achieve a long lasting effect the expansion of the pre-stressing steel has to be greater than elastic compression of the concrete, as the time-dependent plastic deformations have to be compensated. Already before the Second World War two different prestressed systems were designed, which mainly influenced the bridge construction after the war: The internal bounded pre-stressing system, designed by E. Freyssinet and the external unbounded pre-stressing system, developed by Dischinger.

The main advantages of the pre-stressing is that the spans and the slenderness of structures can be increased considerably. Furthermore, the serviceability, mainly the deformations and durability, is improved due to the constant stress distribution throughout the cross-section (Pauser 2002).

The following example show the incredible amount of material that can be saved with these new developments. The bridges that will be compared were built at the same location. The Moselle Bridge near Koblenz-Lützel was completed 1934 with conventional reinforced concrete. For the construction traditional auxiliary structures had to be used. This bridge was then destroyed during the Second World War and therefore had to be rebuilt for the same actions and with the same width. This time, in 1953, the concrete beam was designed as two pre-stressed box girders and the classic cantilever construction method was used. As displayed in Table 2.1 8000 m³ (41%) of concrete, 437 t (31%) of steel (pre-stressing and reinforcing) and 4396 m³ (96%) of wood for scaffolding were saved. Clearly the new methods save an enormous amount of material and subsequently have economic and environmental benefits (Bühler 2004).

*Table 2.1: Comparison of the material used for the Moselle Bridge in 1934 and in 1953* (Bühler 2004)

| | **1934** | **1953** |
|---|---|---|
| **Concrete** | 19500 m³ | 11500 m³ |
| **Reinforced Steel** | 1400 t | 319 t |
| **Pre-stressing Steel** | 0 t | 644 t |
| **Wood for scaffolding** | 4600 m³ | 204 m³ |

### 2.2.1 Historical value of the Bridge by Taubenstein

The bridge by Taubenstein was build 1963 as part of the German federal highway system. The main structure is pre-stressed and the cross-section alters between box girder and T-beam, which are typical for this area (see description above). The designer of the bridge is unknown. Generally, the bridge by Taubenstein is from a historical viewpoint a typical bridge. It was constructed in the ear of the bridge boom in Germany (BMVBS 2013) with the then typical methods and materials.

## 2.3 Aesthetics

In the following the vital aspects of a good design will be explained firstly according to ten guidelines by F. Leonhardt, secondly with various criteria for bridges described by A. Pauser and thirdly with five example of bridges built in the same period as the bridge by Taubenstein. All buildings, which are regarded as beautiful, reveal certain characteristics, such as symmetry, rhythm, repeats, certain proportions and contrasts. These reoccurring features can be used as guidelines for the design. Fritz Leonhardt formulated ten rules in his book "Bridges – Aesthetics and Design" (Leonhardt 1982). The following list gives a short overview of the ten guidelines:

1. Fulfilment of purpose-function

The first and main goal of any structure should be the ideal fulfilment of its purpose. The support structure should always be designed in a clear form to give a sense of stability while always striving for simplicity. Aside from the carrying capacity the structure has to fulfil the serviceability, which limits for example the deflection.

2. Proportion

The structure should have good harmonious proportion in all directions. Considering only statically correctness is insufficient. Not all proportions are important. However for bridges the relation between the suspended main structure and the supporting columns, the relation between the depth and the span of the beam and the relation of the height and width of the openings are crucial. By repeating the same proportions within the bridge the harmony can be created.

3. Order

To establish order within the designed structure the direction of lines has to be limited. Nature gives perfect examples for order through limitations, such as snow crystals. Looking at snow crystals two more effects seem to increase order, symmetry and rhythm through repetition. These two tools can also be implemented into buildings.

4. Refining the form

Bodies with only parallel and straight lines can seem stiff and static. Therefore columns and tower should be tapered or stepped. Using parabolic tapering has a greater impact on refining the form than straight tapering.

5. Integration into the environment

Any structure should always be integrated into the environment, landscape and cityscape. This can be achieved by surface, colour and the dimension of buildings. The dimensions of humans should always be the scale.

6. Surface texture

As mentioned in the fifth guideline the surface texture should fit to the surroundings. The surface is influenced by the choice of material, the texture of the surface and the colour. As a general rule surfaces should always be matt, not glossy.

7. Colour

Colour influences the overall aesthetics. The objective is to strive towards a harmonious colour pallet.

8. Character

Structures should effect the people entering in a different way depending on the purpose of the structure, thus buildings should have a character. A simple example for this is that monarchies build monumental buildings to intimidate the visitors.

9. Complexity

A way to increase beauty can be to enhance the tension between variety and similarity, complexity and order. A small surprising element within very orderly surrounding will be recognised as beauty. On the other hand too much variety it will be overexerting.

10. Incorporating Nature

Another important aspect is the incorporation of nature. Nature should be given a place in manufactured structures.

Fritz Leonhard was a mentor for Alfred Pauser, who received the Fritz-Leonhard award in 2012 (Engelsmann 2012). Like Fritz Leonhard, Alfred Pauser regards civil-engineers to be responsible for designing aesthetical and cultural but at the same time well-engineered and economically sustainable structures (Pauser 1990). The ideal structure according to Alfred Pauser is a simple, precise form without redundant applications. His projects show that the goal of any design should be to reach a point where engineering and art form a single entity (Pauser 1995).

Alfred Pauser defined various criteria for designing bridges in his book “Massivbrücken ganzheitlich betrachtet“ (Pauser 2002). Many are similar to the beforehand described guidelines, as they were conceptualised by his mentor. However as the ten guidelines from Fritz Leonhard regard the general aesthetic of structures, the following criteria focus on bridges.

A selected few criteria concerning aesthetics of bridges are listed in the following:

- The surroundings of the bridge should at least not be devaluated, if they are not improved.
- Aesthetical decisions have to be taken conscientiously and with due diligence if they are connected with disproportionate high costs.
- Shapes have to be explicit as the human eye is able to distinguish between uniformity and differences as well as continuous progression and abrupt changes. However proportions due to small changes are experienced as unsettling.
- The complexity of a bridge structure should not be higher than necessary.
- The proportion between new and known should be balanced.

The following five examples of tensioned concrete bridges will be evaluated with consideration of the before described guidelines and criteria.

The Neckarkanalbrücke in Heilbronn-Böckingen, see Figure 2.7 left, was one of the first long span tensioned concrete beam bridges. It was built in 1950 and has a total span of 96 m. The high slenderness ratio and the simple and precise form give the bridge an elegant impression (Pauser 2002; Leonhardt 1982).

*Figure 2.7: left: Neckarkanalbrücke in Heilbronn-Böckingen; right: Rhine Brigde Bendorf* (Leonhardt 1982)

The Rhine Bridge Bendorf, see Figure 2.7 right, was built 1956 and was one of the first bridges to overcome the span limit of 200 m with the construction method of free cantilevering. The piers seems slender in proportion to the depth of the box beam. This is highlighted by the acute converting ending of the rib. The method of free cantilevering emphasises a decrease in the in girder height towards the middle (Leonhardt 1982).

The Moselle Bridge in Schweich, see Figure 2.8 left, has a span of 192 m and it supported by two thin pier walls on each side. The ribs of the walls are pulled up to the beam to incorporate a visual division of the giant construction. Through the visual separation and the division of the pier into two thinner walls the structure seems more slender, nonetheless the bridge still appears massive in comparison to its surroundings (Leonhardt 1982).

*Figure 2.8: left:: Moselle Bridge, Schweich; right: Main Bridge, Sindlingen* (Leonhardt 1982)

Another possibility is to support the bridge with round columns. This was implemented in the Main Bridge in Sindlingen, see Figure 2.8 right, with a span of 150 m. The thickness of the columns gives a feeling of safety nonetheless the structure doesn't appear too massive due to the slender beam and the shallow haunches. The cantilever width is 5 m on each side makes the construction seem lighter (Leonhardt 1982).

A good solution is the design of the bridge that crosses the Stör River near Itzehoe. The main opening segues smoothly into the ramp bridge with parallel edged beams supported by pairs of free standing columns. The piers of the main openings and the columns have the same angle of taper which brings harmony to the two different parts of the bridge (Leonhardt 1982).

*Figure 2.9: Stör Bridge near Itzehoe* (Leonhardt 1982)

### 2.3.1 Aesthetics of the Bridge by Taubenstein

The main structure and the supports of the highway bridge by Taubenstein have a clear form and the purpose-function is fulfilled. The structure provides a feeling of safety yet the constant height of the girder make the structure seem massive. Haunches, for example, would make the building appear lighter. Similarly to the designs of Rhine Bridge and Moselle Bridge the structure appears more slender through the visual division of the supports. Mainly in the area of the east part of the bridge the supports give the whole structure an impression of elegance, see Figure 2.6 left. On the other hand in the west part of the bridge the supports are less high the opening for the street Spinnereistraße seems compressed, compare Figure 2.5.

## 2.4 Strategy for the Strengthening of existing highway road bridges in Germany

The following chapter is a summary of the report "*Strategie zur Ertüchtigung der Straßenbrücken im Bestand der Bundesfernstraßen*" (BMVBS 2013) published by the BMVBS (Bundesministerium für Verkehr und digitale Infrastruktur) in 2013 and elements of the "*Nachrechnungsrichtlinie*" (Bast 2007).

The main goal of the strategy for the strengthening of existing highway road bridges in Germany is preventing extensive traffic restrictions or blocking of bridges if possible. Guaranteeing a stable and permanent mobility is the main precondition for economic growth and employment and for the living quality of the citizens. This mobility depends highly on the federal highway system. Bridges are next to tunnels the most expensive elements of road systems. The current increase of the traffic load and the age of the structures lead to fast decreasing load reserves. Thus "older bridges" (bridges build before 1985 with the at the time current bridge class BK 60) have to be strengthened for the future.

For these bridges the objective load model according to DIN FB 101 should be aimed for. Additionally the possibility to strengthen to the load model according to DIN EN 1991-2/NA (applied for new structures) should be checked on technical and economic feasibility.

As the highway system includes a vast amount of bridges with the bridge class BK 60, a sequence for their examination is necessary. Seven different selection criteria were defined and at least two of these criteria fit to the bridge by Taubenstein as it is a prestressed concrete bridge built before 1985 and it has multiple spans over 30 m.

The BMVBS defined a maintenance plan for all relevant bridges including a network oriented and object oriented plan. The goal of the network oriented maintenance plan is to optimise the handling of the construction measures in order to minimise the effects on the highway network. For the object-oriented plan feasibility studies including economic and technical aspects have to be conducted.

The necessary funding for the maintenance of the federal highway system is estimated at over 3 billion Euros per year till 2025.

## 2.5 Strategy for the Preservation of Buildings in Switzerland

This chapter is based on the guideline SIA 2017, which gives instructions for determining the maintenance value of a building. On the bases of this value the decision is taken if structures should be preserved.

The worth of a building is a complex value to determine, as it depends on various different criteria. The standard SIA 2017 divides all influence values into two main categories: immaterial or cultural values and material values. The cultural values include the positional value, the historic-cultural value, the aesthetic value, the socio-cultural value and the emotional value. On the other hand, the location, the utilisation, the building material, the society, the economy and the environment are elements defining the material values. Determining the maintenance value of a building has a big influence on the rehabilitation of existing structures, as this step has a significant impact on the further development of a building, all stakeholders should express their views on the value of the building and reach a common value for the structure. This reflection requires good knowledge of the structure nonetheless the results are rarely absolute. A transparent approach simplifies the0 decision making process and consolidates irrational judgments. With this method it can be ensured that on the one hand remarkable works cannot be destroyed and badly replaced and on the other hand structures with very low value can be replaced. With this a long-term suitable solution can be found (SIA 2017 2000; Brühwiler 2014).

In the following the different criteria will be described with the focus on evaluating bridges to lay a foundation for determining the maintenance value of the highway bridge of Taubenstein.

### 2.5.1 Cultural values

#### Positional Value

To define the positional value of the structure its spatial interaction and relationship with the environment have to be analysed. Examples of rating factors are the definition of space, separation of territory and an impressive appearance. Predominantly public works, such as roads, bridges, dams have a high positional value (SIA 2017 2000; Brühwiler 2014).

#### Historic-cultural Value

The historical worth of a structure depends on its economic, political or social standing in an era. Representative structures, which were built with a new building technique, represent a technical development or their original material is irreplaceable, have a high value. Another reason for a high historical value can be the constructer or the resident/user of the building (SIA 2017 2000; Brühwiler 2014).

#### Aesthetic Value

The aesthetic value is mainly defined by architectural qualities, for example the composition, the form of the structure or the uniqueness of the style. Establishing a value for aesthetics is complex as few people have similar opinions and the public conception on aesthetics changes from one generation to the next. Nonetheless aesthetics have a large influence on the overall value of a building. Thus the aesthetic value should be determined with caution (SIA 2017 2000; Brühwiler 2014).

#### Technical Value

The technical value of building depends on the material and the construction methods and technical methods used. The main factors are quality and rarity of the used materials and technical methods and innovative constructions or structures (SIA 2017 2000; Brühwiler 2014).

Socio-cultural Value

Structures for public purposes have a higher socio-cultural value. These buildings give identity, stability and a feeling of security or of well-being to the community (SIA 2017 2000; Brühwiler 2014).

Emotional Value

The emotional value is influenced by the sentimental values, prestige and traditional values. The emotional value can be crucial in the decision making. All stakeholders and the public have objective preferences for or against the preservation of the structure (SIA 2017 2000; Brühwiler 2014).

### 2.5.2 Material values

Location

The value of the location is given by the possibilities of the usage of the plot. Some of the main parameters are the neighbourhood, the terrain, building density, potential dangers such as avalanches or landslides and legal requirements. The value of the location depends not only on constant value but also on unstable factor, for example the neighbourhood (SIA 2017 2000; Brühwiler 2014).

Utilisation

The value of the utilisation is a result of various factors such as the serviceability at the momentary state, the possibility to modify the building or change its function, legal conditions or contracts to consider and the operational safety (SIA 2017 2000; Brühwiler 2014).

Building Material

The value of the building material depends on, amongst other things, the material itself, its state, structural safety and durability, the composition of the structure and the need for rehabilitation. In general different elements of a building have different lifetimes. The main carrying structure has for example a longer durability than the road surface (SIA 2017 2000; Brühwiler 2014).

Society

A building can influence society significantly through, for example, constructions works or downtimes and can thus have a high social value (SIA 2017 2000; Brühwiler 2014).

Economy

The revenue, the utilisation, the insurance value or tax value define the economic value. The cost-benefit analysis should take into account the value of the investment, the cost of mortgages, the maintenance and operations cost, externals costs and the cost of the demolition to compute a realistic value (SIA 2017 2000; Brühwiler 2014).

Environment

The environmental value is defined by various criteria: sustainability, removal and recycling of the building, the land area required and its impact on the environment. (SIA 2017 2000; Brühwiler 2014).

### 2.5.3 Maintenance value of the Bridge by Taubenstein

The following table gives an approach for the derivation of the maintenance value from the viewpoint of a civil engineer. For every value a short explanation, the importance (a) and a grade (b) are given. The importance and the grade can have a value between 1 and 3, where 3 is the highest mark. These numbers are then multiplied to receive a total worth of the value (c).

*Table 2.2: Maintenance value of the bridge by Taubenstein from one stakeholder*

| **Value** | **Explanation** | **a*** | **b*** | **c*** |
|---|---|---|---|---|
| Positional | The positional value of the bridge is high as it is a public structure. | 3 | 3 | 9 |
| Historic-cultural | The bridge is typical for the era therefore the value is low. | 1 | 1 | 1 |
| Aesthetic | The bridge and columns on the west side seem slim and therefore elegant, compare Figure 2.6 right. The bottom view over the river is unique and provides the bridge with more lightness, compare Figure 2.6 left. Even though the structure is not an architectural masterpiece it has unique characteristics and aesthetics aspects. The goal of any intervention measures should be to keep the original design. | 2 | 2 | 4 |
| Technical | The technics used for the construction and the structure are very typical for the era. | 1 | 1 | 1 |
| Socio-cultural | The structure has little socio-cultural value. The bridge is a purpose fulfilling object and maybe gives identity to the community as it is a very distinct building. | 1 | 2 | 2 |
| Emotional | The building is purpose fulfilling and does not have sentimental or traditional values, thus the emotional value is low. | 1 | 1 | 1 |
| Location | The location of the bridge is depended on the existing highway hence the bridge's location is fixed. | 2 | 2 | 4 |
| Utilisation | The utilisation value of the bridge is high as it is part of a highway system thus used frequently. | 3 | 3 | 9 |
| Building Material | The bridge is a massive construction hence a large amount of materials were used. | 2 | 2 | 4 |
| Society | Demolishing and rebuilding the bridge would mean a default for a long period, which would have a huge influence on the society. On the other hand well-grounded intervention would decrease necessary maintenance works thus there would be less traffic disruptions. | 3 | 3 | 9 |
| Economy | The influence on the economy is similar to that on society. Demolition and rebuilding the bridge would entail on the one hand a huge investment and on the other hand long term traffic disruptions involving a potential economic damage. Good interventions entail less maintenance works thus less cost. | 3 | 3 | 9 |
| Environment | The structure has influence on the environment, as it incorporates a large amount of embodied energy. However the demolishing and rebuilding of the bridge would consume even more natural resources and subsequently cause higher pollution. | 1 | 2 | 2 |
| Total | | 55/108 | | |

* a) Importance b) Grade c) Value

Of course this evaluation is subjective and depend on the personal interpretation of each parameter. For a thorough and more accurate calculation, more information and the evaluation from all parties involved, for example neighbours, the public, city officials, the owner or investors would be needed.

Already in this short evolutions it becomes clear that maintaining this existing structure rather than demolishing it and building a new bridge has economical but also social and environmental advantages.

## 2.6 Computation of the traffic loads for existing bridges in the DACH region

The show the differences in the management of existing bridges within the DACH region (Germany, Austria and Switzerland) one aspect, which influence rehabilitation measures largely, will be compared: The computation of the traffic loads for existing bridges. The following comparison focuses on the Load Model 1 (LM1). The LM1 is described in the EN 1991-2 as well as in the SIA 261 and summarised with the Table 2.3.

*Table 2.3: Load model 1* (DIN EN 1991-2 2010; SIA 261 2014)

| | **Tandem System (TS)** $Q_{ik}$ | **Uniform distributed load (UDL)** $q_{ik}$ |
|---|---|---|
| Lane 1 | 300 * $\alpha_{Qi}$ | 9.0 * $\alpha_{qi}$ |
| Lane 2 | 200 * $\alpha_{Qi}$ | 2.5 * $\alpha_{qi}$ |
| Lane 3 | 100 * $\alpha_{Qi}$ | 2.5 * $\alpha_{qi}$ |
| add. lanes | 0 | 2.5 * $\alpha_{qi}$ |
| remaining area | 0 | 2.5 * $\alpha_{qr}$ |

For designing new bridges all three countries apply the Load Model 1. However already for new structures different adjustment factors $\alpha_i$ (factor with which traffic loads are multiplied in order to consider the different road classes and the expected traffic) are used, compare Table 2.4.

*Table 2.4: Different adjustment factors α and partial safety factor according to the three different standards of the DACH region for designing new structures* (ÖNORM B 1991-2 2011; DIN EN 1992-1-1/NA 2013; SIA 261 2014)

| | **AUSTRIA** | **GERMANY** | **SWITZERLAND** |
|---|---|---|---|
| | **ÖNORM B 1991-2** | **DIN EN 1991-2/NA** | **SIA 261** |
| $\alpha_{Qi}$ | $\alpha_{Qi}$ = 1.00 | $\alpha_{Qi}$ = 1.00 | $\alpha_{Qi}$ = 0.9<br>$\alpha_{Q3}$ = 0.0 |
| $\alpha_{qi}$ | $\alpha_{qi}$ = 1.00 | $\alpha_{q1}$ = 1.33<br>$\alpha_{q2}$ = 1.40<br>$\alpha_{q3}$ = 1.20 | $\alpha_{qi}$ = 0.9 |
| $\alpha_{qr}$ | $\alpha_{qr}$ = 1.00 | $\alpha_{qr}$ = 1.20 | $\alpha_{qr}$ = 0.9 |
| γ | 1.50 | 1.35 | 1.50 |

Already this comparison shows differences of more than 10% between the three different standards for the computation of the traffic load.

In case of existing structures the guideline "*Nachrechnungsrichtlinie*" (BMVI 2011) has to be considered for Germany. As described in Section 2.4 the German recalculation guideline refers to the DIN FB 101, which also applies the Load Model 1. The adjustment factors are listed in the following table.

For Switzerland the standard SIA 269 focusing only on existing structures is available. It is also based on the Load Model 1, however it has different adjustment factors than the SIA 261 (Brühwiler et al. 2012; Brühwiler 2011). The Austrian recalculation guideline (ONR 24008) applies different values than the Load Model 1. For an easier comparison the adjustment factors (values of ONR divided by the values of the Load Model 1) are computed.

*Table 2.5: Different adjustment factors $\alpha$ and partial safety factor according to the three different standards of the DACH region for existing structures (ONR 24008 2014; DIN FB 101 2003; SIA 291/1 2011)*

| | **AUSTRIA** | **GERMANY** | **SWITZERLAND** |
|---|---|---|---|
| | **ONR 24008 applied on LM1** | **DIN FB 101** | **SIA 269-1** |
| $\alpha_{Qi}$ | $\alpha_{Q1} = 0.4$<br>$\alpha_{Q2} = 0.6$<br>$\alpha_{Q3} = 1.2$ | $\alpha_{Qi} = 0.8$ | $\alpha_{Qi} = 0.6$ |
| $\alpha_{qi}$ | $\alpha_{q1} = 2.5$<br>$\alpha_{q2} = 3.0$<br>$\alpha_{q3} = 2.6$<br>$\alpha_{q4} = 2.2$ | $\alpha_{qi} = 1.0$ | $\alpha_{qi} = 0.5$ |
| $\alpha_{qr}$ | $\alpha_{qr} = 0.0$ | $\alpha_{qr} = 1.0$ | $\alpha_{qr} = 0.5$ |
| $\gamma$ | 1.35 | 1.50 | 1.50 |

As the regarded bridge is an existing structure situated in Germany the guideline "*Nachrechnungsrichtlinie*" is applied in the following. Additionally, for a comparison, the SIA 269 (Switzerland) will be used for the computation of the traffic loads, in the following section 3 Examination.

# 3 Examination

In this chapter first the necessary information for the examination is given and then the examination of the bridge, divided into the general and detailed examination, is conducted.

## 3.1 Motives and Objectives

The motivation of the examination is to verify the structural resistance of the bridge for the next 50 to 100 years of its utilisation. The aim is to find the most sustainable solution.

## 3.2 Documents and Data

The following documents are available for the examination:

| | |
|---|---|
| *Machbarkeitsuntersuchung zur Ertüchtigung und Wirtschaftlichuntersuchung*<br>*Bauwerk: UF Lahn, L3020 und Stadtstraße Taubenstein* | *Issue: 2013-11* |
| *Bauwerksplan: Draufsicht Überbau und Längsschnitt in Brückenmitte* | *Issue: 2013-10* |
| *Bauwerksplan: Querschnitte – Varianten* | *Issue: 2013-10* |
| *Nachrechnung nach Stufe 1 der Nachrechnungsrichtlinien* | *Issue: 2013-11* |

## 3.3 Standards and Guidelines

The examination is based on the following standards and guidelines:

| | |
|---|---|
| *DIN EN 1991-1-3*<br>*Einwirkungen auf Tragwerke – Teil 1-3: Allgemeine Einwirkung – Schneelasten* | *Issue: 2010-12* |
| *DIN EN 1991-1-3/NA*<br>*Einwirkungen auf Tragwerke – Teil 1-3: Allgemeine Einwirkung – Schneelasten* | *Issue: 2010-12* |
| *DIN EN 1991-2*<br>*Einwirkungen auf Tragwerke – Teil 2: Verkehrslasten auf Brücken* | *Issue: 2010-12* |
| *DIN EN 1991-2/NA*<br>*Einwirkungen auf Tragwerke – Teil 2: Verkehrslasten auf Brücken* | *Issue: 2012-08* |
| *DIN FB 101*<br>*Lasten und Einwirkungen auf Brücken einschließlich Kombinationsregeln* | *Issue: 2009-03* |
| *Nachrechnungsrichtlinie*<br>*Richtlinie zur Nachrechnug von Straßenbrücken im Bestand* | *Issue: 2011-05* |
| *DIN EN 1998-1/NA*<br>*Auslegung von Bauwerken gegen Erdbeben – Teil 1: Grundlagen, Erdbebeneinwirkungen und Regeln für Hochbauten* | *Issue: 2010-12* |
| *DIN EN 1992-1-1*<br>*Bemessung und Konstruktion von Stahlbeton- und Spannbetontragwerken – Teil 1-1: Allgemeine Bemessungsregeln und Regeln für den Hochbau* | *Issue: 2011-01* |
| *DIN EN 1992-1-1/NA*<br>*Bemessung und Konstruktion von Stahlbeton- und Spannbetontragwerken – Teil 1-1: Allgemeine Bemessungsregeln und Regeln für den Hochbau* | *Issue: 2013-04* |
| *SIA 269*<br>*Grundlagen der Erhaltung von Tragwerken* | *Issue: 2011-01* |
| *SIA 269-1*<br>*Erhaltung von Tragwerken – Einwirkungen* | *Issue: 2011-01* |

## 3.4 Programs

The following programs are used for the computations:

| | |
|---|---|
| SOFISTIK 2016 | This static program is used for the interpolation of the cross-section and for the derivation of the internal forces and the reaction forces. Both the model (beam and shell) are designed with this program. |
| FAGUS 7.0<br>Cubus | FAGUS 7.0 is used for the verification of the cross-sections. |

## 3.5 Updating of the Resistance

For the resistance side little information is known therefore the material assumed by *Nachrechnungsstufe 1* (Bösche & Curbach 2013b) is considered. For a more exact determination of the material resistance on-site and laboratory tests would have to be performed.

### 3.5.1 Concrete

For the concrete the strength class C20/25 is considered, according to *Nachrechnungsstufe 1* (Bösche & Curbach 2013b). In the following table the main properties are listed.

*Table 3.1: Properties of the concrete C20/25.*

| **Property** | | **C20/25** | **Unit** |
|---|---|---|---|
| Self-weight | $\gamma$ | 25 | kN/m³ |
| Density | $\rho$ | 2400 | kg/m³ |
| Temperature coeff. | $\alpha$ | $1.0 \times 10^{-5}$ | 1/K |
| Elastic modulus | E | $3.0 \times 10^{4}$ | N/mm² |
| Poisson ratio | $\nu$ | 0.2 | - |
| Shear modulus | G | $1.25 \times 10^{4}$ | N/mm² |
| Nominal strength | $f_{ck}$ | 20 | MPa |
| Effective strength | $f_c$ | 20 | MPa |
| Tensile strength | $f_{ctm}$ | 2.21 | MPa |

### 3.5.2 Reinforcing Steel

For the reinforcing steel the strength class Bst 550 is assumed. The following table lists the main properties.

*Table 3.2: Properties of the reinforcing Steel Bst 550.*

| **Property** | | **Bst 550** | **Unit** |
|---|---|---|---|
| Self-weight | $\gamma$ | 78.5 | kN/m³ |
| Temperature coeff. | $\alpha$ | $1.2 \times 10^{-5}$ | 1/K |
| Elastic modulus | E | $2.0 \times 10^{5}$ | N/mm² |
| Poisson ratio | $\nu$ | 0.3 | - |
| Shear modulus | G | $7.7 \times 10^{4}$ | N/mm² |
| Yield strength | $f_y$ | 550 | MPa |
| Compressive yield strength | $f_{yc}$ | 550 | MPa |

### 3.5.3 Pre-stressing Steel

For the longitudinal direction pre-stressing steel with the classification ST150/170 is assumed for the calculation. In transversal direction pre-stressing steel with the classification Leoba S 33 is taken in account. The data for the pre-stressing steel is taken from *Nachrechnungsstufe 1* (Bösche & Curbach 2013b) and the main properties are listed below.

*Table 3.3: Properties of pre-stressing steel ST150/170 or Leoba S33.*

| Property | | ST150/170 | Leoba S 33 | Unit |
|---|---|---|---|---|
| Self-weight | $\gamma$ | 78.5 | 78.5 | kN/m³ |
| Temperature coeff. | $\alpha$ | $1.2 \times 10^{-5}$ | $1.2 \times 10^{-5}$ | 1/K |
| Elastic modulus | E | $2.1 \times 10^{5}$ | $2.0 \times 10^{5}$ | N/mm² |
| Poisson ratio | $\nu$ | 0.3 | 0.3 | - |
| Yield strength | $f_y$ | 1470 | 1320 | MPa |
| Ultimate strength | $f_u$ | 1665 | 1470 | MPa |

## 3.6 Geometry

For the general information about the bridge's geometry, see section 2.1, page 3.

## 3.7 Updating of actions

The computations of the actions (forces and loads) is performed on the base of the German recalculation guideline, which refers to both the DIN EN 1991 and the DIN FB 101. In case of the traffic load two different scenarios are regarded: The first scenario is computed with values according only to the DIN FB 101 and the second scenario with the updated values according to SIA 269-1.

### 3.7.1 Self-weight

The computation of the bridge's self-weight results from the exact interpolation of variable cross-sections over the bridge's length with the program Sofistik. The exact dimensions of 94 cross-sections used for the interpolation are given in Appendix A. The load is compared with manual calculation to verify the computation and the difference is below 10%. The total self-weight of the bridge is around 105 MN according to the interpolation with Sofistik, when only considering the superstructure of the bridge without any additional loads.

Due to the changing cross-section throughout the length of the bridge the self-weight is not constant. The distribution is shown on Figure 2.7.

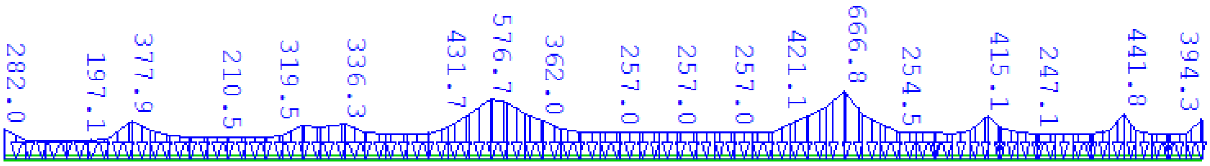



*Figure 3.1: Loads due to self-weight of the deck without the compression slab*

The partial safety factor of $\gamma_G$=1.2 is considered (BMVI 2011; SIA 291/1 2011)

### 3.7.2 Additional dead load

For the additional permeant loads the following have to be considered:

*Table 3.4: Additionally considered permeant loads*

| Loads to be considered: | Computation | Final Value |
|---|---|---|
| Road surface with waterproofing | $g_{2.a.k}$ = 0.11 m x 24 kN/m³ | 2.64 kN/m² |
| Railings | $g_{2.b.k}$ = | 0.40 kN/m |
| Shoulder Curb | $g_{2.k}$ = 0.47 m² x 25 kN/m³ /1.45 m | 8.10 kN/m² [11.75 kN/m] |

As the additional dead load is a permanent load the partial safety factor ( $\gamma_{G2} = 1.2$ ) is the same as for the self-weight. The total additional dead load is equal to 25 MN, around one quarter of the self-weight of the bridge.

### 3.7.3 Pre-stressing

The pre-stressing will be considered on the resistance side. The area $A_p$ for one tendon is equal to 1244 mm² (Bösche & Curbach 2013b). The number of tendons differs across the length of the bridge and also between inner and outer T-Beams. The following Figure 3.2 shows the total number of tendons and their average eccentricity from the top of the beam for the most relevant cross-sections (mid-field and support). From the amount of tendons it can be expected that the bridge has complete pre-stressing typical for this era (60s/70s), which means that the necessary pre-stress is computed with 100% of the service load.

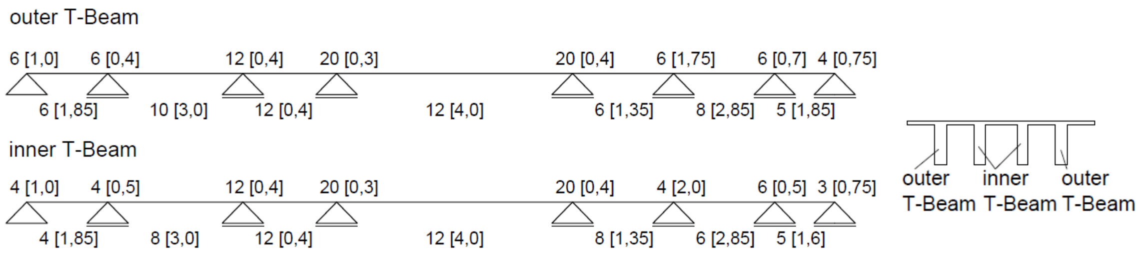



*Figure 3.2: Number of tendons and eccentricity from the top in brackets [m] for the main cross-sections*

With the ultimate tensile strength $f_{pk}$ and the dimension of the tendons the pre-stress forces can be computed. It is assumed that the losses of the force are 20%.

| | | | |
|---|---|---|---|
| $f_{pk}$ | | 1665.0 | N/mm² |
| $f_{p0,1k}$ | 0.9 x $f_{pk}$ | 1498.5 | N/mm² |
| $f_{pd}$ | $f_{p0,1k}$/1.15 | 1303.0 | N/mm² |
| $F_p$ | $f_{pd}$ x $A_p$ | 1620.0 | kN |
| $\sigma_{pm,0}$ | min = 0,7 x $f_{pk}$ = 1165.5 N/mm²<br>0,8 x $f_{p0,1k}$ = 1198.8 N/mm² | 1165.5 | N/mm² |
| $P_{t0}$ | $\sigma_{pm,0}$ x $A_p$ | 1449.9 | N/mm² |
| $P_\infty$ | $P_{t0}$ (1-0.2) | 1159.9 | N/mm² |
| $n_{py}$ | $(F_p - P_\infty)/(A_p\, E_p)$ | 0.0185 | ‰ |

### 3.7.4 Snow load

The snow load is dependent on altitude of the building location (160 m above mean sea level), the snow load zone (Zone 2) and the inclination of the structure (nearly 0%). Thus the snow load for the bridge by Taubenstein is 0.68 kN/m² (DIN EN 1991-1-3 2010; DIN EN 1991-1-3/NA 2010).

The snow load will not be considered for the computation of the ULS for mainly two reasons. Firstly, the snow load is minimal in comparison to the traffic load and additionally it is minimised with the multiplication of the combination coefficient as the snow load is never the dominant load. Secondly, the bridge never has the maximal snow load and heavy traffic loading as the snow has to be cleared to ensure the road safety. Therefore, the value for the snow load is considered to be negligible.

### 3.7.5 Traffic loads

For the traffic loads the following load cases have to be considered: Load Model 1 tandem system and Load model 1 uniform distributed loads and the horizontal loads. Both loads of the LM1 have to be regarded at the same time. To input the loads into the model the carriageway with 15.1 m has to be divided into lanes with a width of 3.0 m according to the DIN EN 1991-2 Tab. 4.1, therefore there are 5 lanes. Two assumption are taken here: Firstly the retaining system that divides the two direction is removable therefore the complete carriageway is taken into account. Secondly the carriageway becomes wider but as the number of lanes stays constant, the carriageway is assumed with 15.1 m for the complete length.

*Table 3.5: Considered load cases for the traffic* (DIN FB 101 2003; DIN EN 1991-2 2010)

| | | | |
|---|---|---|---|
| $g_{r1a}$ | Load Model 1 – Tandem System | Lane 1<br>Lane 2<br>Lane 3<br>Lane 4<br>Lane 5 | 300 kN<br>200 kN<br>100 kN<br>0 kN<br>0 kN |
| $g_{r1b}$ | Load Model 1 –<br>Uniform Distributed Loads | Lane 1<br>Lane 2<br>Lane 3<br>Lane 4<br>Lane 5 | 9.0 kN/m²<br>2.5 kN/m²<br>2.5 kN/m²<br>2.5 kN/m²<br>2.5 kN/m² |
| $g_{r2}$ | Horizontal loads | | |

As mentioned before two scenarios are considered for the LM1, compare section 2.6. The first scenario uses the DIN FB 101 and the second the SIA 269-1. The adjustment factors are given in the table below.

*Table 3.6: Different adjustment factors $\alpha$ for the two different scenarios* (DIN FB 101 2003; SIA 291/1 2011)

| | **DIN FB 101** | **SIA 269-1** |
|---|---|---|
| $\alpha_{Qi}$ | 0.8 | 0.6 |
| $\alpha_{qi}$ | 1.0 | 0.5 |
| $\alpha_{qr}$ | 1.0 | 0.5 |

The "*Nachrechungsrichtlinie*" only defines which adjustment factor should be used. All other information is taken from the current standard (DIN EN 1991-2). According to DIN EN 1991-2 these different loads have to be inputted onto the model in the most unfavourable way. Therefore two different cases were computed for every load model and the programme SOFISTIK automatically generates the internal force envelope for the most unfavourable case.

According to the DIN EN 1991-2/NA the Load Model 2 and the Load Model 3 do not have to be applied. The centrifugal force and the forces due to breaking will be considered in all traffic load cases. The Traffic Load Manager of Sofistik calculates the centrifugal force with the velocity $v = 100\ km/h$ and the geometry of the bridge, radius of $r = 450\ m$. For the break load the maximal load of 900 kN is considered (DIN EN 1991-2/NA 2012; DIN EN 1991-2 2010).

### 3.7.6 Temperature

The influence of the temperature will not be considered for the computation of the ULS as temperature induced deformations do not generate internal stress. This is due to the fact that the bridge can deform in longitudinal direction.

The temperature will be taken into consideration to determine the maximal longitudinal elongation. For this the maximal temperature difference of 60 K (maximal temperature of 40 °C and minimal temperature of -20 °C) will be assumed.

### 3.7.7 Wind forces

The wind forces on the bridge and on trucks (which induce a moment on the road) have to be considered, see Figure 3.3. As the bridge is situated in an urban area the wind load can be assumed to be $w_c = 1.0\ kN/m^2$. The wind onto the structure itself will be applied onto the side of the carriageway slab (W2 + W3 in Figure 3.3). The wind load onto the truck will be entered over the Traffic Load Manager of Sofistik. The assumed wind force is $w_c = 1.0\ kN/m^2$ and the height of the truck is 4.0 m, see Figure 3.3.

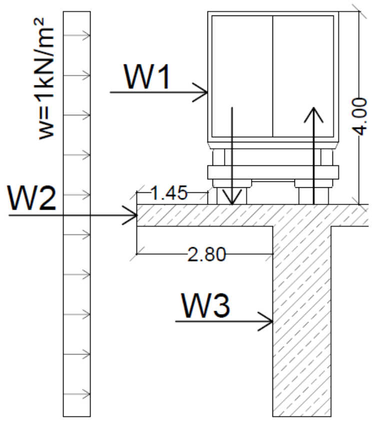



*Figure 3.3: Wind forces to consider on the structure*

### 3.7.8 Accidental Action - Earthquake

The DIN EN 1998-1/NA distinguishes only three design relevant earthquake zones in Germany. Outside of these three zones the seismic risk for buildings is assessed sufficiently low that a verification of the seismic safety is not necessary. The bridge by Taubenstein lies outside the three relevant zones and therefore an earthquake safety evaluation is not necessary (DIN EN 1998-1/NA 2010).

### 3.7.9 Accidental Action - Impact

The accidental action impact has to be considered for the columns of the bridge as there are two streets passing under the bridge. For this, two scenarios have to be examined: impact force in travel direction and perpendicular to the travel direction. The static forces are positioned at a height of 1.25 m according to DIN EN 1991-2. Ship collisions with the columns of the bridge are not relevant as the piles are not situated in the river bay (DIN EN 1991-2 2010).

| | | |
|---|---|---|
| Impact force in travel direction | $A_{i1}$ | 1000 kN |
| Impact force across travel direction | $A_{i2}$ | 500 kN |

## 3.8 Load Combinations

### 3.8.1 Combination Coefficients

*Table 3.7: Combination Coefficients for live loads (DIN EN 1991)*

| **Load** | | $\psi_0$ | $\psi_1$ | $\psi_2$ |
|---|---|---|---|---|
| $g_{r1a}$ | LM1 – Tandem System | 0.75 | 0.75 | 0 |
| $g_{r1b}$ | LM1 – UDL | 0.4 | 0.4 | 0 |
| $g_{r2}$ | horizontal loads | 0 | 0 | 0 |
| w | Wind | 0.6 | 0.2 | 0 |

### 3.8.2 Characteristic Load Combinations – Ultimate Limit State

| | $\sum(G_{k,i} \times \varphi_{G,i}) \oplus Q_{k,1} \times \varphi_Q \oplus \sum_{(i\geq 2)} Q_{k,i} \times \varphi_{Q,i} \times \psi_{0,i}$ |
|---|---|
| LC 1 | $(g_1 + \sum g_{2.i})\ x\ 1.2 + (g_{r1a}\ x\ \alpha_{Qi}\ 1.0 + g_{r1b}\ x\ \alpha_{qi}\ x\ 1.0)\ x\ 1.5 + (w\ x\ 0.2)\ x\ 1.5$ |
| LC 2 | $(g_1 + \sum g_{2.i})\ x\ 1.2 + (g_{r3}\ x\ 1.0)\ x\ 1.5 + (g_{r1a}\ x\ \alpha_{Qi}\ x\ 0.75 + g_{r1b}\ x\ \alpha_{qi}\ x\ 0.4\ w\ x\ 0.2)\ x\ 1.5$ |
| LC 3 | $(g_1 + \sum g_{2.i})\ x\ 1.2 + (w\ x\ 1.0)\ x\ 1.5 + (g_{r1a}\ x\ \alpha_{Qi}\ x\ 0.75 + g_{r1b}\ x\ \alpha_{qi}\ x\ 0.4)\ x\ 1.5$ |

$\oplus$ = has to be combined with

For the first load case combination the leading action is the traffic load of LM1, for the second the horizontal load and for the third the wind forces.

### 3.8.3 Characteristic Load Combinations – Ultimate Limit State – Impact

| | $\sum(G_{k,i}) \oplus A_d \oplus \sum_{(i\geq 2)} Q_{k,i} \times \psi_{2,i}$ |
|---|---|
| LC I-1 | $g_1 + \sum g_{2,i} + A_d$ |

$\oplus$ = has to be combined with

### 3.8.4 Load Combinations – Serviceability Limit State

The serviceability for existing structures according to SIA 269 only needs to be confirmed if the utilisation of the structure has changed (e.g.: higher live loads). Otherwise if the utilisation of the structure remained the same, the serviceability merely has to be verified on the basis of the condition survey of the momentary condition. The condition survey of the bridge by Taubenstein (*Nachrechnungsstufe 1*) shows that the serviceability is not affected. As a simplification the SLS and the fatigue behaviour will not be studied.

For a more exact analysis the German recalculation guideline would have to be used. The serviceability would have to be evaluated computationally and qualitatively. For the computational evaluation the decompression, the tension limitation and crack width have to be calculated according to DIN FB 102. Deformations and oscillations do not have to be analysed. For the qualitative evaluation the actual cracking behaviours have to be examined on site. Additionally, the fatigue has to be analysed and evaluated (BMVI 2011).

## 3.9 Model

As a general simplification the height difference in the horizontal (less than 1%) and longitudinal (less than 3%) directions are minimal and will therefore not be implemented into the models.

### 3.9.1 Beam Model

The beam model is a simple model of beams with a slab (T-Beam with four webs) with an altering cross-section throughout the length of the bridge. As a simplification, the compression slab is not included in the beam model. This simple model is used to analyse the bridge in the longitudinal direction, in a first approach, and for the validation of the results for the shell model. The results and computation of the beam model can be found in the Appendix B.

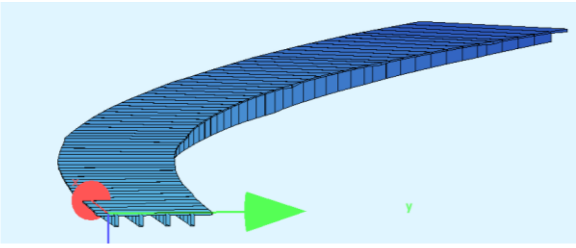


*Figure 3.4: Beam model – 3D view (Sofistik)*

For this two-dimensional system all the supports - expect the support at axis 1 [$x = 0\ m$] – are modelled as plain bearings, which only bear vertical loads. The support at axis 1 also takes on horizontal loads. The static system in longitudinal direction is a continuous beam.

### 3.9.2 3D Shell Model for longitudinal analysis

The shell model is based on the beam model and has two additional elements. Firstly, the roadway slab is modelled as an orthotropic plate, which decreases local deformations and enables the structure to support and forward punctual loads better.

Secondly, the compression slab is implemented, as seen in Figure 3.5. Therefore in some areas the cross-sections are box girders with three cells, which increase the stiffness of the system in these areas. The distribution of the supports is shown in Figure 3.5.

The 3D Shell Model is necessary as it is more realistic and the derived results are therefore more accurate. The following examination is computed on the shell model.

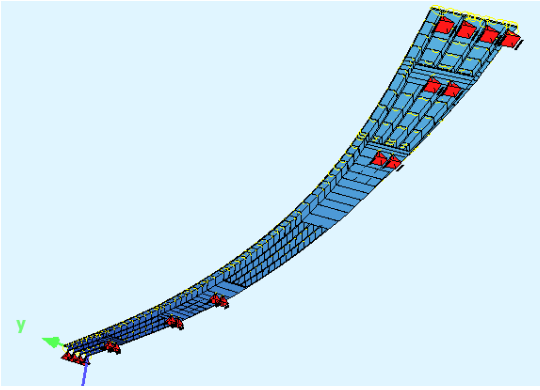



*Figure 3.5: Bottom view of the shell model displaying the different cross-sections (Sofistik)*

### 3.9.3 3D Shell Models for the transversal analysis

To analyse the bridge in transversal directions the two most extreme cases, the shortest and the longest span, are regarded. The shortest span of 23.75 m (between Axis 7 – Axis 8) and longest span of 93.35 m (between Axis 4 – Axis 5) are analysed separately on two different models, as seen in Figure 3.6.

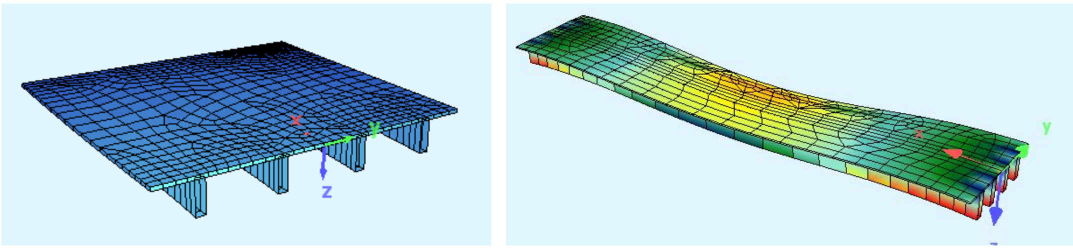



*Figure 3.6: left: Shell model of the area with the shortest span; right: Deformation due to self-weight of the shell model of the area with the longest span (Sofistik)*

The results are simulated with clamped supports and with hinged supports. The average values of both extreme cases are then regarded for the verification for a first simplified approach. The real static system of the bridge is situated between these extremes but for a detailed analysis the transversal effects would have to be regarded over the complete bridge.

## 3.10 General Assumptions for the Calculations

In the following sections the assumptions for the calculation are summarised.

### 3.10.1 Degree of compliance $n$

The verification of the bridge will be computed with the degree of compliance $n$, computed by the following equation.

$$n = \frac{R_{d,upd}}{E_{d,upd}} \geq 1 \tag{1}$$

The structural safety is verified if the updated resisting forces ($R_{d,upd}$) are greater than the updated actions ($E_{d,upd}$).

### 3.10.2 Regarded Cross-section for the longitudinal analyse

For the verifications in longitudinal direction the four most crucial cross-section are analysed. The positions of the four cross-sections are shown in Figure 3.7.

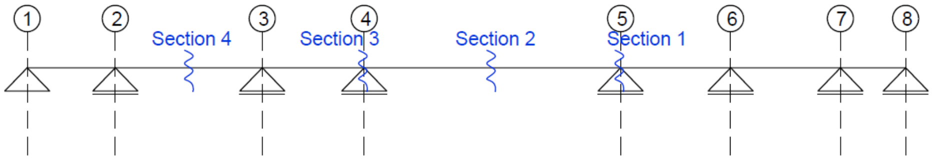



*Figure 3.7: Schematic longitudinal cut of the bridge with the four decisive cross-sections*

The following table shows the position of the sections along the length of the bridge and the main dimension for the cross-sections. The Figure 3.8 explains the different values.

*Table 3.8: Main geometric data for the four cross-sections*

| Sect. | | Long. Coordinate [m] | $h_f$ [mm] | $b_w$ [mm] | $A_a$ [m] | $A_i$ [m] | C Cantilever [m] | H [m] |
|---|---|---|---|---|---|---|---|---|
| 1 | Support | 216.0 | 900 | 1150 | 3.9 | 2.2 | 2.8 | 4.4 |
| 2 | Midfield | 169.3 | 200 | 400 | 3.8 | 2.1 | 2.8 | 4.4 |
| 3 | Support | 122.7 | 830 | 1150 | 3.8 | 2.1 | 2.8 | 4.4 |
| 4 | Midfield | 58.8 | 200 | 400 | 3.8 | 2.1 | 2.8 | 3.2 |

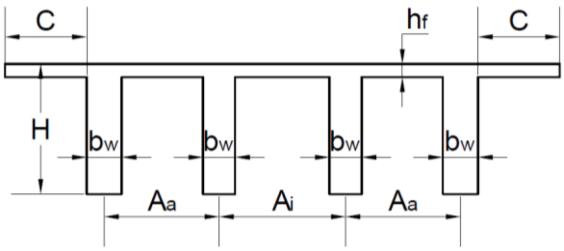



*Figure 3.8: Exemplary cross-section*

For the cross-sections at the supports (Section 1 and Section 3) the height $h_f$ is the height of the compression plate.

### 3.10.3 Minimal Reinforcement

As there is no information about the existing reinforcement, it is assumed that at least the minimal reinforcement is built-in. In the following the minimal reinforcement is derived.

The following equation (DIN EN 1992-1-1 2011) is used to compute the minimal reinforcement for bending and normal force:

$$A_{s,min} = \frac{M_{CR}}{0.9\, d\, f_{sd}} = \frac{w_{elastic}\, f_{ct}}{0.9\, d\, f_{sd}} \tag{2}$$

*Table 3.9: Minimal reinforcement for bending in longitudinal direction*

| Section | $I_y$ [m⁴] | $z_s$ [m²] | $W_{elastic}$ [m³] | $f_{ct}$ [MN/m²] | $\sigma_{sd}$ [MN/m²] | d [m] | $A_{smin}$ [cm²] | $A_{smin}$ per beam [cm²] |
|---|---|---|---|---|---|---|---|---|
| 1 | 82.78 | 2.17 | 38.15 | 2.21 | 220.00 | 4.20 | 1013.8 | 253.4 |
| 2 | 21.48 | 2.88 | 7.46 | 2.21 | 220.00 | 4.20 | 198.2 | 49.6 |
| 3 | 66.19 | 1.98 | 33.43 | 2.21 | 220.00 | 4.20 | 888.4 | 222.1 |
| 4 | 8.96 | 2.18 | 4.11 | 2.21 | 220.00 | 3.00 | 152.9 | 38.2 |

The same equation (see equation (2)) is used to derive the minimal reinforcement for bending in transversal direction.

*Table 3.10: Minimal reinforcement for bending in transversal direction*

| Section | b [m] | h [m] | $I_y$ [m⁴] | $z_s$ [m] | $w_{inf}$ [m³] | $f_{ct}$ [MN/m²] | $f_{sd}$ [MN/m²] | d [m] | $A_{smin}$ [cm²/m] |
|---|---|---|---|---|---|---|---|---|---|
| Axis 4-5 | 1.0 | 0.283 | 0.002 | 0.14 | 0.01 | 2.21 | 220 | 0.25 | 5.8 |
| Axis 7-8 | 1.0 | 0.288 | 0.002 | 0.14 | 0.01 | 2.21 | 220 | 0.26 | 6.0 |

For the shear the following minimal reinforcement is considered. As the width of the webs in the area of the supports (Section 1 and Section 3) is equal to 1.1 m (>0.5 m) 4-legged stirrups are necessary. However for the two other sections (Section 2 and Section 4) 2-legged stirrups are sufficient.

*Table 3.11: Minimal reinforcement for shear*

| Section | $A_{sw}$ | $A_{sw}$ per element [cm²/m] |
|---|---|---|
| 1 | Ø16/150 | 53.6 |
| 2 | Ø14/150 | 20.5 |
| 3 | Ø16/150 | 56.3 |
| 4 | Ø14/150 | 20.5 |

The minimal reinforcement for the piles is considered with the following approximation
(DIN EN 1992-1-1 2011):

$$A_{s,min}[cm^2] = A_c * 0{,}0026\ [cm^2] \tag{3}$$

*Table 3.12: Minimal reinforcement of the piles*

| | b [m] | h [m] | A [m²] | As [cm²] |
|---|---|---|---|---|
| Axis 2 | 1.2 | 1.2 | 1.44 | 37.44 |
| Axis 3 | 1.1 | 1.1 | 1.21 | 31.46 |
| Axis 5 | 1.4 | 1.4 | 1.96 | 50.96 |
| Axis 7 | 1.3 | 1.3 | 1.69 | 43.94 |

### 3.10.4 Current Deformations

No data is available on the current deformation of the structure and according to *Nachrechnungsstufe 1* nearly no cracks are visible. Thus for the following computations it will be assumed that there are no existing deformations at the current point that may have an impact on the structural behaviour (for example long-term effects like creeping or tension loss of the prestressed reinforcement).

## 3.11 General Examination

The following examination is computed on the shell model.

### 3.11.1 General Examination in longitudinal direction

For the general examination in longitudinal direction the actions described in section 3.7 “Updating of actions” are applied onto the model. For the bending and axial the internal forces $N$, $M_y$ and $M_z$ are considered and for the shear forces and torsion the internal forces $V_y$, $V_z$ and $M_T$.

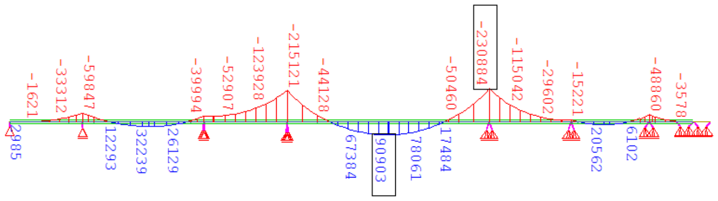



*Figure 3.9: Internal force distribution of the moment $M_y$ due to self-weight [kNm]*

To establish the most unfavourable load case combination LC 1, LC 2 and LC 3 are calculated for all six internal forces. LC 1 is the most unfavourable load case combination. The results for the LC 1 are displayed in the Table 3.13 for the computation of the traffic loads with DIN FB 101 and in Table 3.14 for the computation with the traffic loads according to SIA 269/1. The complete table with all load combinations is attached in Appendix C.

*Table 3.13: Internal forces – LC 1 – DIN*

| Sec. | N [kN] | $M_y$ [kNm] | $M_z$ [kNm] | $V_y$ [kN] | $V_z$ [kN] | $M_t$ [kNm] |
|---|---|---|---|---|---|---|
| 1 | -18590 | -594037 | 274101 | -8428 | -31373 | -31373 |
| 2 | 18355 | 378168 | -166777 | 3032 | 2559 | 2559 |
| 3 | -17660 | -595094 | -239397 | -7469 | 31430 | 31430 |
| 4 | 17925 | 217208 | -198443 | 9440 | 4566 | 4566 |

*Table 3.14: Internal forces – LC 1 – SIA*

| Sec. | N [kN] | $M_y$ [kNm] | $M_z$ [kNm] | $V_y$ [kN] | $V_z$ [kN] | $M_t$ [kNm] |
|---|---|---|---|---|---|---|
| 1 | -13375 | -520199 | 197154 | -6546 | -29418 | -18545 |
| 2 | 13074 | 306684 | -118550 | 2122 | 1791 | 10135 |
| 3 | -12446 | -512361 | -180694 | -5283 | 29389 | -22983 |
| 4 | 12549 | 167222 | -159931 | 7660 | 3567 | 14062 |

These values can be reduced due to two different reasons. The values of the bending moment $M_y$ can be redistributed as the supports in the model are idealised as points. In reality, the support have a certain width. The moment $M_y$ can be average out over the area of the supports, as seen in Figure 3.10.

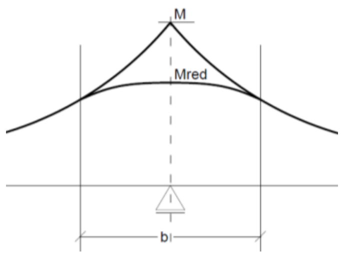



*Figure 3.10: Reduction of $M_y$*

In case of the shear force and torsion, the EC 1992-1-1 states that values at the supports can be reduced by a factor depending on the angle of the strut $cot(\theta)$ and the statically effective height $d$.

The cross-section verification is computed with FAGUS 7.0 and the utilisation factors are summarised in Table 3.15 for the extreme and reduced values.

*Table 3.15: Degree of compliance n computed with FAGUS 7.0 with the reduced actions*

| | Bending and axial force | | Shear forces and torsion | |
|---|---|---|---|---|
| **Sec.** | **DIN n [-]** | **SIA n [-]** | **DIN n [-]** | **SIA n [-]** |
| 1 | 1.12 | 1.32 | 0.86 | 0.93 |
| 2 | 0.75 | 0.93 | 2.04 | 2.94 |
| 3 | 1.10 | 1.28 | 0.85 | 0.93 |
| 4 | 0.74 | 0.95 | 0.93 | 1.23 |

### 3.11.2 General Examination of the Shell in transversal direction

To verify the shell in transversal direction the bending moment $m_x$ in x-direction is analysed. The maximal resisting moment is calculated with the following formula (DIN EN 1992-1-1 2011):

$$x_B = \frac{A_{s,min}\, f_{yd}}{d\, f_{cd}} \tag{4}$$

$$m_{Rd} = d\ x_B\ f_{cd}\,(d - 0.5\, x_B) \tag{5}$$

*Table 3.16: Resisting Moment for bending in x-direction and degree of compliance n*

| | | | DIN | | | | SIA | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | **xb [m]** | **$m_{rd}$ [kNm/m]** | **$m_{x,ed,\,max}$ [kNm/m]** | **$m_{x,ed,\,min}$ [kNm/m]** | **n [-]** | **n [-]** | **$m_{x,ed,\,max}$ [kNm/m]** | **$m_{x,ed,\,min}$ [kNm/m]** | **n [-]** | **n [-]** |
| Axis 4-5 | 0.02 | 70.74 | 83.90 | -54.50 | 0.84 | 1.30 | 74.10 | -50.40 | 0.95 | 1.40 |
| Axis 7-8 | 0.02 | 70.74 | 35.45 | -52.10 | 1.99 | 1.36 | 32.85 | -49.40 | 2.15 | 1.43 |

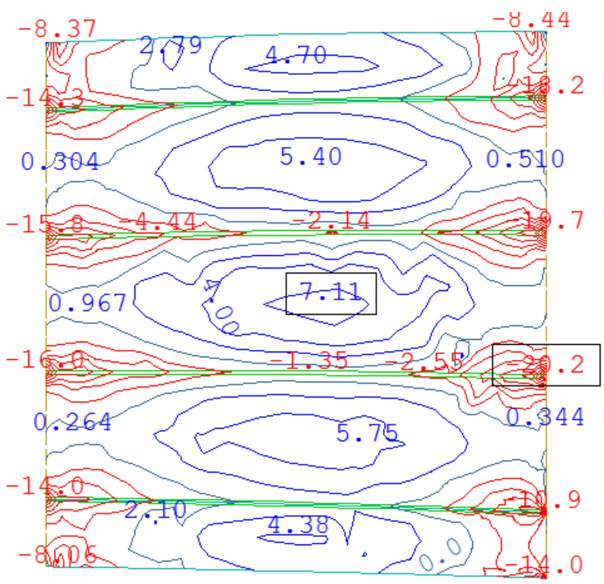



*Figure 3.11: Distribution of the bending moment $m_x$ on the model for the area with the shortest span [kNm/m]*

### 3.11.3 General Examination of the Piles

For the verification of the piles two cases have to be regarded – one case for the normal loads and one for the accidental action due to impact. The support reactions ($N_x$ and $M_y$) of the model are used for the forces acting on the piles. The static system of the piles is idealised with the third Euler mode.

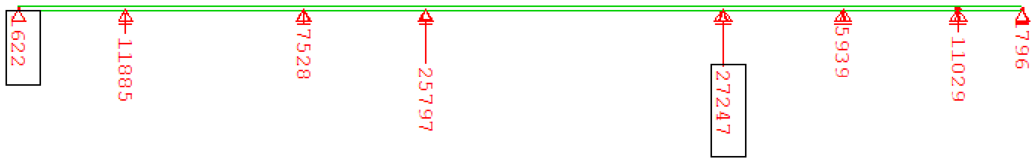



*Figure 3.12: Support reaction due to self-weight of the bridge*

Two exemplary piles with the most unfavourable geometry and loads will be verified, axis 2 and axis 5. The load case combination LC 1 and LC 3 will be analysed to derive which load case combination produces the highest moment. For the accidental load case the columns on axis 2, 3 and 7 are relevant for the examination of the collision with a truck.

For both cases the moment due to the fictitious eccentricity according to DIN EN 1992-2 is computed by hand, compare Table 3.17. The values for $N_x$ and $M_y$ (see values highlighted in blue in Table 3.17) are then imported into FAGUS 7.0 were the utilisation factor with the minimal reinforcement (ρ=0.5%) and the M-N-Interaction diagrams are designed.

*Table 3.17: M due to fictitious eccentricity and degree of compliance n for all examined cases*

| | Axis 2 | | | Axis 5 | | Axis 3 | Axis 7 | Unit |
|---|---|---|---|---|---|---|---|---|
| | LC-1 | LC-3 | LC-A | LC-1 | LC-3 | LC-A | LC-A | |
| Ned | 16062 | 13465 | 43645 | 27594 | 24803 | 5388 | 4311 | kN |
| Med | 202 | 1011 | 1250 | 193 | 965 | 1250 | 1250 | kNm |
| $e_{0,y}$ | 0.013 | 0.075 | 0.286 | 0.007 | 0.04 | 0.23 | 0.29 | m |
| $e_{1,y}$ | 0.025 | 0.025 | 0.025 | 0.015 | 0.015 | 0.02 | 0.02 | m |
| $e_{2,y}$ | 0.020 | 0.020 | 0.020 | 0.020 | 0.020 | 0.02 | 0.02 | |
| $e_{0,z}$ | 0.020 | 0.020 | 0.020 | 0.02 | 0.02 | 0.02 | 0.02 | m |
| $e_{1,z}$ | 0.018 | 0.018 | 0.018 | 0.0105 | 0.0105 | 0.01 | 0.01 | m |
| $e_{2,z}$ | 0.020 | 0.020 | 0.020 | 0.020 | 0.020 | 0.02 | 0.02 | |
| $e'_y$ | 0.10 | 0.17 | 0.38 | 0.08 | 0.11 | 0.32 | 0.37 | m |
| M' | 1.66 | 2.24 | 1.65 | 2.27 | 2.84 | 1.70 | 1.58 | MNm |
| n - FAGUS | 1.35 | 1.41 | 2.56 | 1.14 | 1.19 | 1.89 | 3.03 | - |

The following diagrams show the M-N-Interaction for the piles on the axis 2 for all three relevant load case combinations: Load case combination 1, load case combination 3 and load case combination for accidental loading. All three cases are within the $\rho = 0.5\%$ border. Thus in all three cases the forces $M_y$ and $N_x$ can be endured, as already shown with the utilisation factor in Table 3.17.

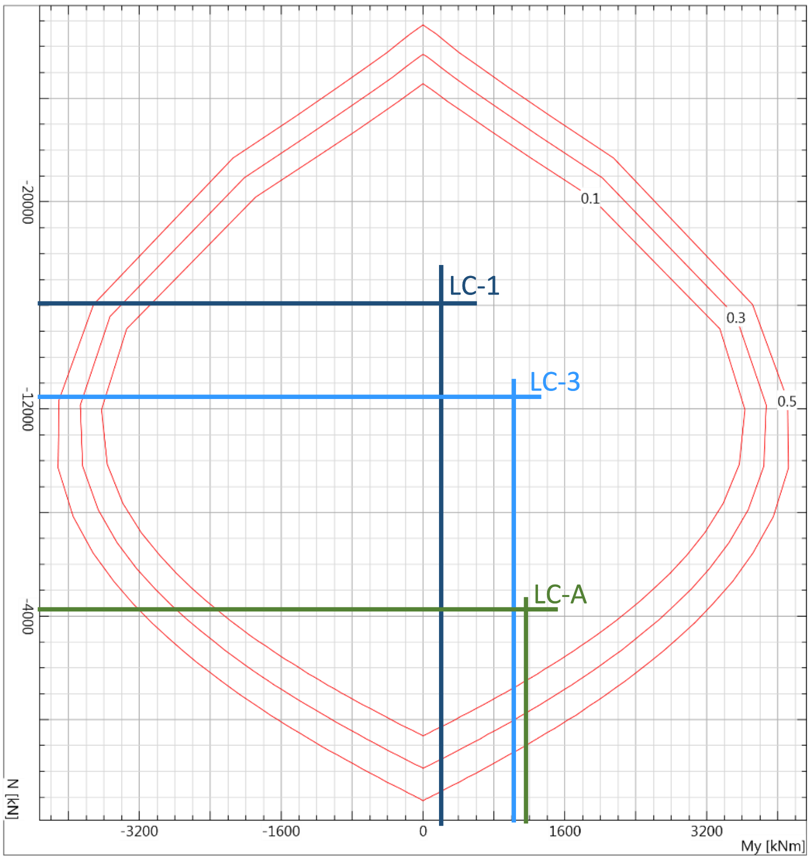



*Figure 3.13: M-N-Interaction diagram for the pile on axis 2*

### 3.11.4 Deformations

In the following the possible deformation of the system are computed. For comparison and for validation of the model, the deflection due to self-weight is computed by hand. The formula (7) is a simply approximation for the deflection of a continuous beam.

$$I_y = \frac{1}{12}\,(B\,H^3 - b\,h^3) = \frac{1}{12}\,(16.2 \times 4.4^3 - (2 \times 2.4 \times 4.2^3 + 3 \times 3.4 \times 4.2^3)) = 22.4\,m^4 \tag{6}$$

$$w = \frac{M\,l^2}{9.6\,E\,I} = \frac{91 \times 93.35^2}{9.6 \times 2.996 \times 10^4 \times 22.4} = 0{,}123\,m \tag{7}$$

The difference between the maximal deflections computed by hand or by SOFISTIK is due to simplifications applied when calculating the deflection manually. The moment of inertia and as well as the used formula are simplifications.

*Table 3.18: Maximal deflections computed with Sofistik*

| Action | | Maximal Deflection | |
|---|---|---|---|
| G | Self-weight | 142 mm | The maximal deflection due to self-weight is in the midfield of the biggest span. |
| T | Temperature | 150 mm | The maximal deflection in longitudinal direction due to a temperature difference of 60 K. |
| w | Wind | 0.2 mm | The maximal deflection over the whole length of the bridge due to wind in transversal direction. |

The computed deflections are low and fit to the assumption on the current deformation. For an exact analysis the current deformation would have to be taken on the existing structure to verify these results.

### 3.11.5 Summary and discussion of the results of the general examination

In the following the results of the general examination are summarised to show in which areas a more detailed examination is necessary. The bending moments ($M_y$ and $M_z$) and axial forces can be absorbed in the cross-section 1 and 3 (at the supports). In mid-field, the bending moments and axial force exceed the resistance by 5% or 7% in the case of the traffic load computation according to SIA 269/1 and with over 20% in the case of traffic load computation according to DIN FB 101, as seen on the next table.

*Table 3.19: Degree of compliance n for the deck – reduced bending moments and axial force*

| | **SIA n [-]** | |
|---|---|---|
| Section 1 | 1.32 | ≥ 1.0 |
| Section 2 | 0.93 | ≥ 1.0 |
| Section 3 | 1.28 | ≥ 1.0 |
| Section 4 | 0.95 | ≥ 1.0 |

| | **DIN n [-]** | |
|---|---|---|
| Section 1 | 1.12 | ≥ 1.0 |
| Section 2 | 0.75 | ≥ 1.0 |
| Section 3 | 1.10 | ≥ 1.0 |
| Section 4 | 0.74 | ≥ 1.0 |

The shear forces and torsion are easily absorbed in midfield according to the load case combination with SIA on the other hand in the area of the supports (Section 1 and Section 3) the present forces exceed the resisting forces. When using the reduced shear forces and torsion and the traffic load computed with SIA 269/1 the acting forces exceed the resisting forces by less than 10%.

*Table 3.20: Degree of compliance n for the deck – reduced shear forces and torsion*

| | **SIA n [-]** | |
|---|---|---|
| Section 1 | 0.93 | ≥ 1.0 |
| Section 2 | 2.94 | ≥ 1.0 |
| Section 3 | 0.93 | ≥ 1.0 |
| Section 4 | 1.23 | ≥ 1.0 |

| | **DIN n [-]** | |
|---|---|---|
| Section 1 | 0.86 | ≥ 1.0 |
| Section 2 | 2.04 | ≥ 1.0 |
| Section 3 | 0.85 | ≥ 1.0 |
| Section 4 | 0.93 | ≥ 1.0 |

The degree of compliance $n$ is not fulfilled for the bending moment $m_x$ in the Axis 4-5.

*Table 3.21: Degree of compliance n of the bending moment $m_x$*

| | **SIA n [-]** | |
|---|---|---|
| Section 1 | 1.40 | ≥ 1.0 |
| Section 2 | 0.95 | ≥ 1.0 |
| Section 3 | 1.43 | ≥ 1.0 |
| Section 4 | 2.15 | ≥ 1.0 |

| | **DIN n [-]** | |
|---|---|---|
| Section 1 | 1.30 | ≥ 1.0 |
| Section 2 | 0.84 | ≥ 1.0 |
| Section 3 | 1.36 | ≥ 1.0 |
| Section 4 | 1.99 | ≥ 1.0 |

The piles are only verified with values of the traffic load due to DIN FB 101. As these higher values are already validated, it is not necessary to validate the lower values.

*Table 3.22: Degree of compliance n for the piles*

| | LC-1 | LC-3 | LC-A | |
|---|---|---|---|---|
| | [-] | [-] | [-] | |
| Axis 2 | 1.35 | 1.41 | 2.56 | ≥ 1.0 |
| Axis 3 | - | - | 1.89 | ≥ 1.0 |
| Axis 5 | 1.14 | 1.19 | - | ≥ 1.0 |
| Axis 7 | - | - | 3.03 | ≥ 1.0 |

## 3.12 Detailed Examination

As the bending moment in the transversal and longitudinal directions and the shear force do not satisfy the structural safety, these areas have to be regarded in more detail. The detailed examination is based on the general examination however it also entails non-linear approaches.

### 3.12.1 Bending in longitudinal direction

As described before, the axial-bending in mid-field exceed the resistance, however, the theory of plasticity based on the plastic hinge analysis can be applied, which allows a redistribution of the moments shown in Table 3.23. The percentage of the possible redistribution is only limited by the degree of compliance in the area of the supports. In this case a redistribution of up to 10% is possible for the computation with the DIN FB 101 and a redistribution of up to 15% with SIA.

*Table 3.23: Degree of compliance n for the deck – reduced bending moments and axial force – Plasticity theory*

| | SIA n [-] | |
|---|---|---|
| Section 1 | 1.32 | ≥ 1.15 |
| Section 2 | 0.93 | ≥ 0.85 |
| Section 3 | 1.28 | ≥ 1.15 |
| Section 4 | 0.95 | ≥ 0.85 |

| | EC n [-] | |
|---|---|---|
| Section 1 | 1.12 | ≥ 1.1 |
| Section 2 | 0.75 | ≥ 0.9 |
| Section 3 | 1.10 | ≥ 1.1 |
| Section 4 | 0.74 | ≥ 0.9 |

As the four regarded cross-sections are the most unfavourable cross-section, it is safe to assume that redistribution is possible throughout the span of the bridge. The structural safety is satisfied according to the standard SIA, however it cannot be fulfilled when using the German recalculation guideline (*Nachrechnungsrichtlinie*).

### 3.12.2 Bending in transversal direction

Similarly to the bending moments in longitudinal direction, the theory of plasticity can be considered for the bending in transversal direction. In this case the forces the structural safety is fulfilled when the plasticity theory is applied.

*Table 3.24: Degree of compliance n of the bending moment $m_x$ – Plasticity theory*

| | SIA n [-] | |
|---|---|---|
| Section 1 | 1.40 | ≥ 1.15 |
| Section 2 | 0.95 | ≥ 0.85 |
| Section 3 | 1.43 | ≥ 1.15 |
| Section 4 | 2.15 | ≥ 0.85 |

| | EC n [-] | |
|---|---|---|
| Section 1 | 1.30 | ≥ 1.15 |
| Section 2 | 0.85 | ≥ 0.85 |
| Section 3 | 1.36 | ≥ 1.15 |
| Section 4 | 1.99 | ≥ 0.85 |

### 3.12.3 Shear forces and torsion

The DIN EN 1992 states that the angle of the strut $cot\,(\theta)$ can be chosen freely within a certain range (e.g. 1.0 ≤ $cot\,(\theta)$ ≤ 2.5). The national annexes then define this range for each country. For Germany (DIN EN 1992/NA) the range is set between 4/7 ≤ $cot(\theta)$ ≤ 7/4 and the equation for the calculation of the angle of the strut is based on a truss model with crack friction (DIN EN 1992-1-1 2011; DIN EN 1992-1-1/NA 2013).

$$\frac{4}{7} \leq \cot\theta = \frac{\cot\beta_r}{1-\frac{V_{Ed}}{V_{Rd,c}}} = \frac{1.2-1.4\frac{\sigma_{cp}}{f_{cd}}}{1-\frac{V_{Ed}}{V_{Rd,c}}} \leq \frac{7}{4} \tag{8}$$

$$V_{Rd,c} = 0.24\ f_{ck}^{\frac{1}{3}} * \left(1-1.2\ \frac{\sigma_{cp}}{f_{cd}}\right)\ b_w\ z \tag{9}$$

With the angle of the strut the shear capacity $V_{Rd,s}$ dependent on the shear reinforcement can be calculated, compare equation (10). The complete shear force is in this case composed of the shear force in z- and y-direction and the torsion.

$$V_{Rd,s} = a_{sw}\ z\ f_{ywd}\ cot\ \theta \tag{10}$$

*Table 3.25: Degree of compliance for the shear and torsion* (DIN EN 1992-1-1/NA 2013)

| | | $V_{Ed,total}$ [kN] | A [m²] | $N_{Ed}$ + P [kN] | $\sigma_{cp}$ [MN/m²] | $V_{rd,c}$ [MN] | $V_{ed}$ [MN] | cot θ [-] | $V_{rd,s}$ [MN] | n [-] | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Sect. 1 | EC | 35204 | 34,63 | 18590 | 1.47 | 12.99 | 35.20 | 0.61 | 25.07 | 0.71 | ≥ 1.0 |
| | SIA | 31620 | 34,63 | 13375 | 1.32 | 12.83 | 31.62 | 0.72 | 29.73 | 0.94 | ≥ 1.0 |
| Sect. 3 | EC | 34761 | 34,63 | 17660 | 1.45 | 12.96 | 34.76 | 0.62 | 25.56 | 0.74 | ≥ 1.0 |
| | SIA | 30773 | 34,63 | 12446 | 1.30 | 12.81 | 30.77 | 0.76 | 31.10 | 1.01 | ≥ 1.0 |

As the structural safety is still not satisfied, the equation for the derivation of the angle of the strut can be regarded in higher detail. Research has shown the angle of the shear crack $\beta_r$ is not only depended on the longitudinal stresses $\sigma_{cp}$ (see equation (8)) but also on the shear reinforcement ratio (see part of equation (11) in bold). The following equation can be used for the verification of older road bridges in Germany (Teworte et al. 2015; Bast 2007).

$$\cot\beta_r = 1.2 + \frac{\boldsymbol{f_{cd}}}{\boldsymbol{70\ \rho_w\ f_{ywd}}} - 1.4\ \frac{\sigma_{cp}}{f_{cd}} \leq 2.25 \tag{11}$$

$$\frac{4}{7} \leq \cot\theta = \cot\beta_r + \frac{V_{Rd,c}}{(A_{sw}/s)\ z\ f_{ywd}} \leq\ 2.5 \tag{12}$$

*Table 3.26: Degree of compliance for the calculation with the provision for older bridges (Teworte et al. 2015; Bast 2007)*

| | | cot $\beta_r$ [-] | cot θ [-] | $V_{rd,s}$ [MN] | n [-] | |
|---|---|---|---|---|---|---|
| Section 1 | EC | 1.70 | 2.02 | 82.90 | 2.35 | ≥ 1.0 |
| | SIA | 1.69 | 2.00 | 82.21 | 2.60 | ≥ 1.0 |
| Section 3 | EC | 1.71 | 2.02 | 82.99 | 2.39 | ≥ 1.0 |
| | SIA | 1.84 | 2.16 | 88.41 | 2.87 | ≥ 1.0 |

With the approach for the provision for older road bridges, considering not only the longitudinal stresses but also the shear reinforcement ratio, the structural safety is fulfilled for all sections independent on the standard uses for the deviation. The following diagram shows the importance of an accurate and detailed computation of the angle of strut considering all effects, as it has a big influence on the shear capacity and therefore on the degree of compliance.

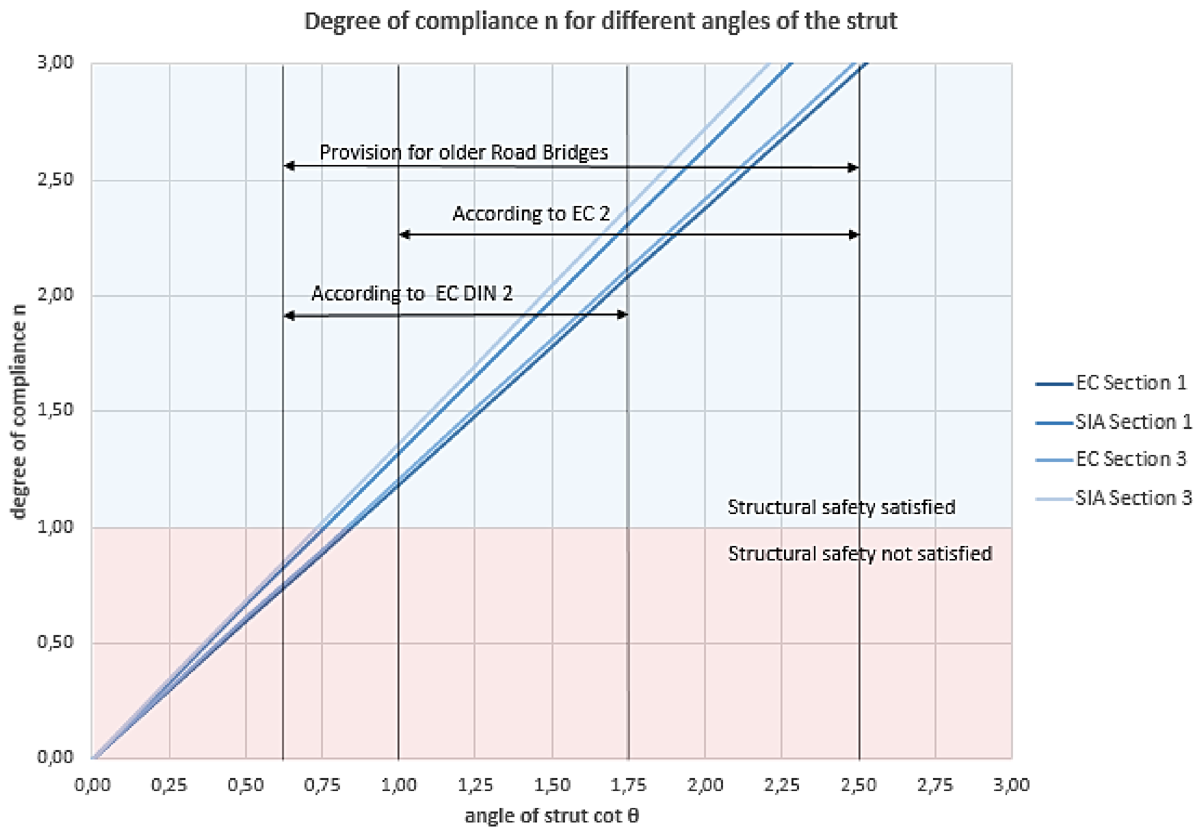



*Figure 3.14: Significance of angle of strut for the degree of compliance*

### 3.12.4 Summary and discussion of the results of the detailed examination

This short detailed examination shows clearly the importance of a closer analysis of the problematic areas. For bending it is important to consider not only the elastic theory but also the plasticity theory. And in the case of verification for shear an exact calculation of the angle of strut has a substantial influence – in this case positive – on the shear resistance. The Table 3.27 summarises the results of the detailed examination.

*Table 3.27: Summary of the detailed examination*

| Traffic Load calculated with | DIN FB 101 | SIA 269 |
|---|---|---|
| Bending – longitudinal | not satisfied | satisfied |
| Bending – transversal | satisfied | satisfied |
| Shear | satisfied | satisfied |

## 3.13 Conclusion and Recommendation

The computation of the degree of compliance clearly shows the importance of choosing the correct adjustment factor for the computation of the traffic loads, as according to SIA 269 the bridge is verified in its present state. For this case no intervention would be necessary. However as the bridge is situated in Germany the Eurocode has to be taken into account or the German recalculation guideline can be applied (which was done in this examination). When taking the German recalculation guideline in account the structural safety is not satisfied for the bending of the deck in longitudinal direction and an intervention is necessary, compare the summary in Table 3.28.

*Table 3.28: Summary of the condition of the different elements*

| Element | Internal force | Conclusion |
|---|---|---|
| Deck | Bending (longitudinal) | Intervention necessary |
| Deck | Bending (transversal) | OK |
| Deck | Shear and Torsion | OK |
| Deck | Deflections | OK |
| Piles | Bending moment and Normal force | OK |

In case of using only the Eurocode the degree of compliance would be even lower, as the adjustment factor $\alpha$ according to DIN EN 1991-2 are above 1. However as the bridge is an existing structure the German recalculation guideline, which refers to the DIN FB 101, should be applied.

# 4 Concepts of Intervention

In the following three intervention concepts are introduced. The first two are the strengthening of the structure with either UHPFRC or CFRP and the last concept uses a different approach to solving the problem. Instead of increasing the structural resistance, the acting loads are reduced.

In this chapter all results and assumptions considered in the Examination (Chapter 3, page 15) are adopted. In the following only the scenario with the traffic loads according to the German recalculation guideline will be analysed for the strengthening interventions, as no strengthening is necessary in the scenario with the SIA 269-1. The scenario with the traffic loads according to SIA 269-1 will be used as a comparison for the third concept.

In the following sections, the conceptual idea of the intervention and the material properties are described for the two strengthening methods (UHPFRC and CFRP). This is followed by dimensioning of the intervention, the verification of the structural safety and the effects of the intervention on the bridge's durability. Additionally, the planned implementation are shown schematically. In the last section, an intervention through decreasing the acting loads is discussed, focusing on the durability and structural safety of the bridge.

## 4.1 Intervention with UHPFRC

The conceptual idea of using Ultra-high Performance Fibre Reinforced cement-based Composites (UHPFRC) is to apply the material as a thin layer over the existing bridge deck. The top surface of the bridge deck is a zone of severe mechanical and environmental exposure. In order to increase the load carrying capacity a layer of reinforced UHPFRC is applied. This additional layer can be regarded as externally bonded additional reinforcement (Brühwiler et al. 2005).

### 4.1.1 Properties of UHPFRC

Ultra-high Performance Fibre Reinforced cement-based Composite is a composite material consisting of cement, additives, fine aggregate, water, admixtures and short fibres. The following image displays the composition of the material in comparison to concrete.

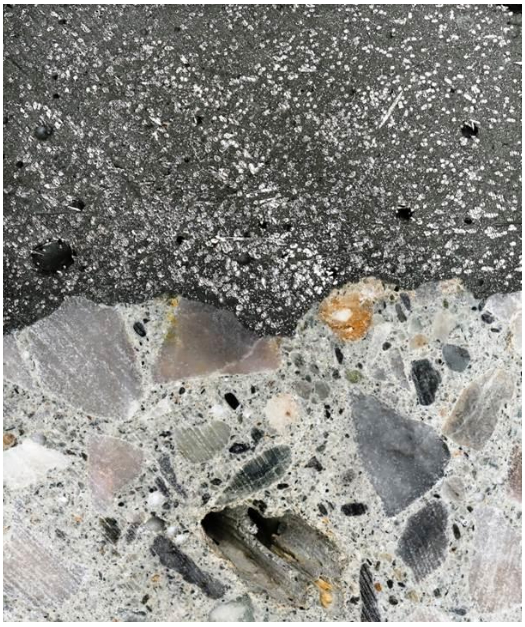


*Figure 4.1: A layer of UHPFRC on top of an existing layer of concrete to display the differences in the composition of the materials* (Brühwiler)

The aggregates are mainly smaller than 1 mm and the packaging density is high and optimised. Due to the elevated density the material is impervious to fluids which obstructs the deterioration of concrete. During the hardening of the concrete the water is consumed completely and thus no capillary pores are created, which minimises the entry of water into the concrete to a negligible degree. The difference between conventional fibre concrete and UHPFRC are mainly the fine aggregates, the higher percentage of fibres, the high packaging density and therefore the high mechanical properties (SIA 2052 2015). The compressive cube strength of UHPFRC after 28 days is generally higher than 120 MPa and the tensile strength is higher than 10 MPa (Brühwiler & Denarié 2013).

In general UHPFRC is used in either new, primarily prefabricated structures or for the rehabilitation or reinforcement of existing structures. In the following paragraphs, only the usage for rehabilitation or reinforcement will be discussed. The material is mainly used for existing reinforced concrete structure and by applying UHPFRC onto the existing structure a composite structure is created (SIA 2052 2015).

In order to apply the material in an economically efficient manner, UHPFRC should only be applied in zones, where its properties are fully exploited (Brühwiler & Denarié 2013), as the material itself is expensive. UHPFRC can, for example, be used in areas with severe environmental conditions, residual stress or high mechanical strain. Bridges are ideal structure for the usage of the material as they are always under high mechanical strain due to the traffic and severe environmental conditions, for example road salt. UHPFRC is mostly used for zones with the following conditions:

- Exposure classes XD2b, XD3, XF4 and chemical attacks
- High mechanicals strain due to temporary or constant design situation
- High mechanicals strain due to extraordinary impacts (e.g. explosion, earthquake)
- High mechanical strain due to abrasion (due to traffic)

When the material is used for rehabilitation it is generally used in combination with the conventional reinforced concrete to decrease the expenses, thus creating a composite material (SIA 2052 2015). The UHPFRC used in the intervention has the following properties (SIA 2052 2015):

*Table 4.1: Mechanical properties of UHPFRC*

| Property | | UHPFRC | Unit |
|---|---|---|---|
| Self-weight | $\gamma$ | 25 | kN/m³ |
| Elastic modulus | E | 50 | N/mm² |
| Nominal strength | $f_{Uc}$ | 160 | MPa |
| Tensile strength | $f_{Utu}$ | 11 | MPa |
| Fracture strain | $\varepsilon$ | 2.5 | ‰ |
| Poisson ratio | $\nu$ | 0.2 | - |

Additionally the UHPFRC will be reinforced with Ø10/100 mm in the transversal direction and Ø10/100 mm in the longitudinal direction ($A_{total}$=$1571\ \mathrm{mm}^2/\mathrm{m}$). The reinforcements significantly improve the tensile behaviour of UHPFRC and reduce the adverse effects of material scatter. The reinforcement provides in-plane continuity and increases the resistance, the deformation capacity as well as the strain hardening behaviour of UHPFRC (Brühwiler 2014).

### 4.1.2 Dimensioning and verification of the structural safety

The most effective thickness is computed through a short analysis with FAGUS 7, as seen in Figure 4.2. An increase of the self-weight in taken into account on the acting side and an increase of resistance on the resisting side. Both the sections in midfield (Section 2 and Section 4) with the insufficient degree of compliance and the sections at the supports (Section 1 and Section 3) are regarded with varying thickness of UHPFRC and additional reinforcement with an area 1571 mm²/m.

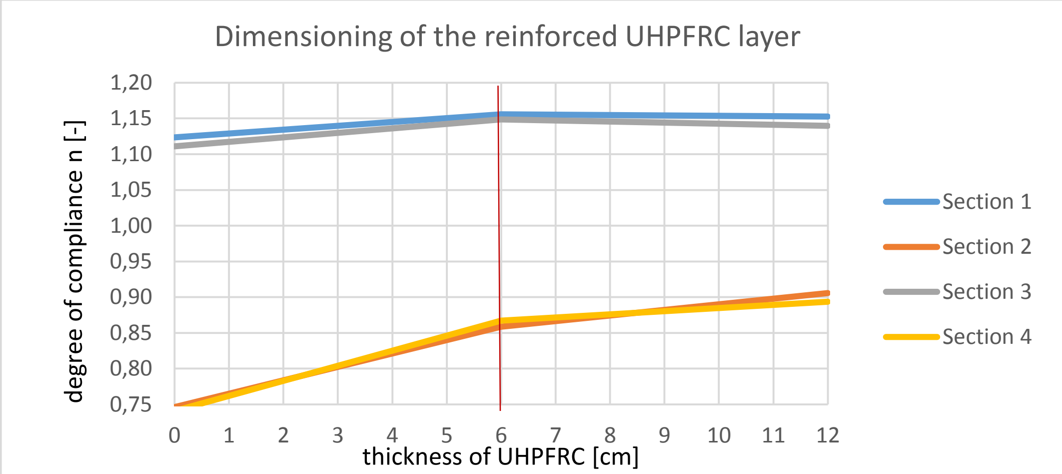



*Figure 4.2: Diagram for the dimensioning of the UHPFRC layer*

A layer with a thickness of 6 cm is selected, as it has the optimal effect. In the following table the degree of compliance for each section with the additional layer of reinforced UHPFRC (R-UHPFRC) is summarised. Due to the theory of plasticity a redistribution of forces is allowed, as seen in section 3.12.1 on page 30. With the application of UHPFRC up to 15% can be distributed, as the degree of compliance in the area of the supports has increased to over 1.15. With the redistribution of 15% the structural safety is verified for all sections as the four regarded cross-section are the most unfavourable cross-sections, compare Table 4.2.

*Table 4.2: Degree of compliance n for the bending with the additional layer with 6 cm of reinforced UHPFRC*

| | **n [-]** | |
|---|---|---|
| Section 1 | 1.16 | ≥ 1.15 |
| Section 2 | 0.86 | ≥ 0.85 |
| Section 3 | 1.15 | ≥ 1.15 |
| Section 4 | 0.87 | ≥ 0.85 |

The resisting moment with the additional layer of UHPFRC is computed in order to verify the bridge deck in the transversal direction with the additional layer (added self-weight). When regarding a distribution of the forces, the deck is verified in transversal direction with the additional layer, as seen in Table 4.3.

*Table 4.3: Degree of compliance n of the bending moment $m_x$*

| | **[-]** | |
|---|---|---|
| Axis 4-5 (min) | 1.34 | ≥ 1.15 |
| Axis 4-5 (max) | 0.87 | ≥ 0.85 |
| Axis 7-8 (min) | 1.41 | ≥ 1.85 |
| Axis 7-8 (max) | 2.07 | ≥ 0.15 |

The shear forces and torsion will not be analysed as the degree of compliance $n$ is large and has reserves of over 100% (section 0), making the exact computation unnecessary for this simplified analysis.

A recalculation of the piles can also be neglected as the additional self-weight only increases the compression and is minimal in comparison to the self-weight of the original structure.

*Table 4.4: Summary of the examination with the intervention*

| Traffic Load calculated with | DIN FB 101 |
|---|---|
| Bending – longitudinal | satisfied |
| Bending – transversal | satisfied |
| Shear | satisfied |
| Piles | satisfied |

In order to protect and preserve the existing structure the complete deck is covered with UHPFRC, compare the following schematic sketch.

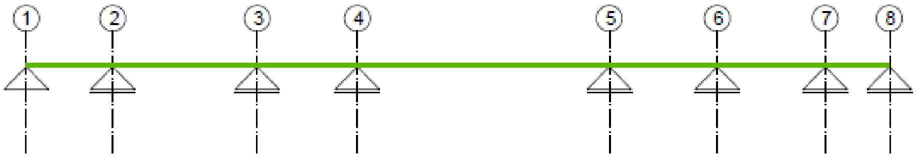



*Figure 4.3: The areas of the deck where UHFPRC is applied marked in orange*

The necessary volume is computed in order to compute and compare the environmental and economic impact.

$$V_{UHPFRC,nec} = 18\,m \times 0.06\,m \times\ 319.75 = 345.33\,m^3 \tag{13}$$

$$V_{Steel,nec} = 1571\frac{mm^2}{m} \times 15\,m \times 319.75\,m = 7.53\,m^3 \tag{14}$$

### 4.1.3 Effect of the intervention with UHPFRC on the durability

The durability of cementitious materials is primarily defined by their permeability due to material composition or cracks. Thus, long-term durability can only be obtained with low permeability and no macro-cracks. UHPFRC has an extremely low permeability thus preventing the infiltration of, for example, chloride-containing water. Experiments have shown that the water permeability of UHPFRC with a tensile deformation of 0.15% is similar to that of uncracked concrete. By adding a layer of UHPFRC the crack formation is controlled and an impermeable layer protects the existing concrete layer. Additionally, the layer between the rebars and the environment is increased thus reducing the likelihood of water infiltration and a destruction of the passivation layer (Brühwiler & Denarié 2013; Charron et al. 2007; Brühwiler et al. 2005).

With these properties of UHPFRC and the experiments conducted on it, it can be assumed that the remaining service life of the bridge after the intervention is at least another 50 years without any major future interventions. Thus a complete life time of around 100 years would be achieved. However as UHPFRC is a new material and the oldest application is only 10 years old, no data on the durability is available yet.

### 4.1.4 Implementation

As described in section 4.1.2 UHPFRC will be applied over the complete bridge deck. Before the application, the concrete surface will be destroyed with a hydro demolition. Through this a rough surface with protruding rebars is created. Any loose particles or composite diminishing substances have to be removed. The necessary surface roughness is 3 to 5 mm. The created increased friction between the existing structure and the new material is mandatory for the cohesion of the materials. Cohesion between the materials is the condition for the applicability and the complete reaping of the benefits of the UHPFRC (SIA 2052 2015).

The following schematic plans show the planned intervention on the bridge.

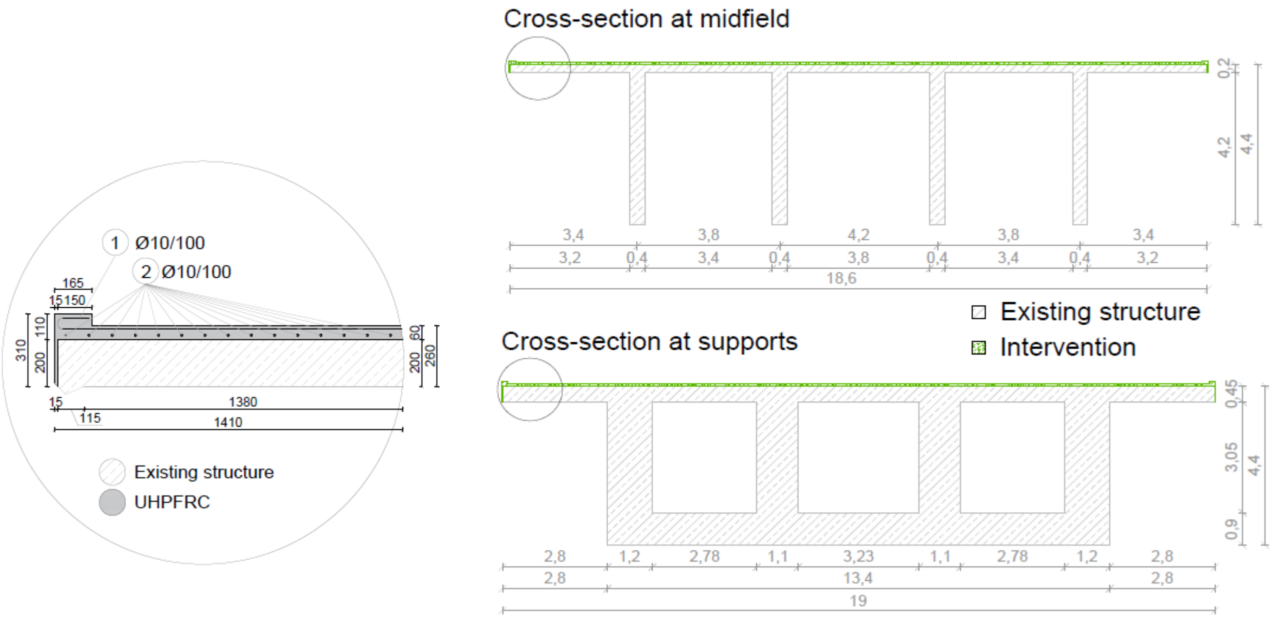



*Figure 4.4: Implementation Plan UHPFRC [m], Detail [mm]*

The minimal necessary reinforcement cover $c_{nom}$ for UHPFRC structures is 15 mm, on the side of framework $c_{nom}$ can be reduced to 10 mm (SIA 2052 2015).

## 4.2 Intervention with CFRP

The conceptual idea of an intervention with CFRP lamella is the strengthening of structure, mainly a flexural strengthening, with external elements. These carbon-fibre-reinforced polymer lamella (CFRP) are glued to the main structure (SIKA 2014).

### 4.2.1 Properties CFRP

Carbon-fibre-reinforced polymer lamella have the advantages of having a high stiffness, low self-weight, a high fatigue resistance durability and the advantage of not corroding. For the structural strengthening, in this case flexural strengthening, very thin lamellae are used, therefore little space is consumed and weight of the structure is hardly increased (SIKA 2014). The disadvantages is mainly the cost of the product from an environmental and ecological perspective. For the intervention concept the material CarboDur M from the company Sika is considered. In the following table the properties of the lamellae are listed (SIKA 2014).

*Table 4.5: Properties of CarboDur M*

| Property | | | Unit |
|---|---|---|---|
| Self-weight | γ | 15,7 | kN/m³ |
| Density | ρ | 1600,0 | kg/m³ |
| Elastic modulus | E | 210,0 | kN/mm² |
| fracture strain | ε | 1,7 | % |
| Tensile strength | $f_t$ | 3500,0 | MPa |

### 4.2.2 Dimensioning and verification of the structural safety

The CFRP laminates will only be applied in the area of the midfield, as these are the areas with inefficient structural safety according to the German guideline. The computation of the degree of compliance for different sizes of laminates is computed with the loads according to DIN FB 101 with FAGUS 7 and displayed in the following diagram.

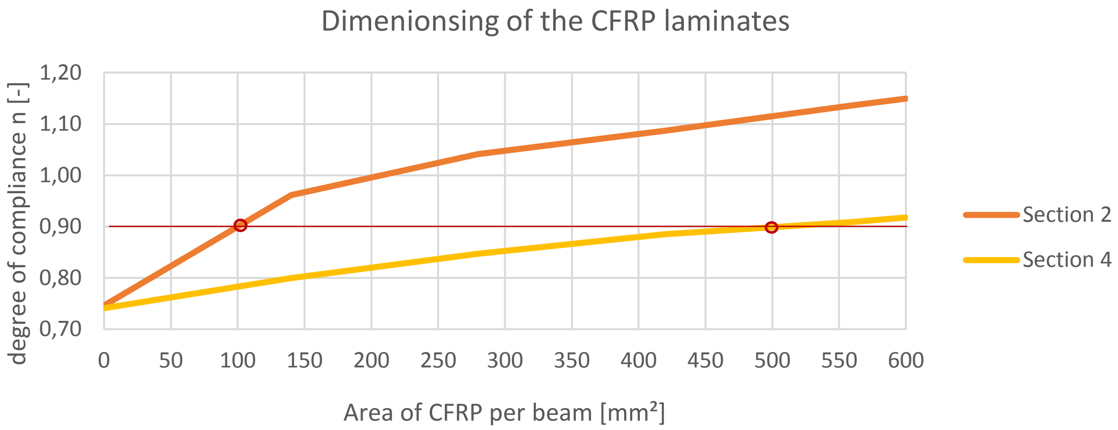



*Figure 4.5: Diagram for the dimensioning of the CFRP laminates*

The degree of compliance n has to be above 1, compare equation (1). However due to the theory of plasticity (described in section 3.12.1, page 30) a redistribution of forces is allowed. As the degree of compliance is above 1.1 in the area of the supports, the degree of compliance only has to be above 0.9 for the sections in midfield. Therefore 100 mm² of CFRP per beam would be necessary for the section 2 and 500 mm² for section 4, see Figure 4.5. The positive effects of the CFRP laminate on the structural safety is bigger for section 2 as the static height is higher than that of section 4. The sections in the area of the supports are not regarded as the application of CFRP laminates in midfield has minimal effect.

The recalculation of the piles, as well as the bending moment in transversal direction and the shear forces and torsion are negligible as the additional self-weight ($g=0.018\,\mathrm{kN/m}$) is minimal in comparison to the self-weight of the original structure.

*Table 4.6: Summary of the examination with the intervention*

| Traffic Load calculated with | DIN FB 101 |
|---|---|
| Bending – longitudinal | satisfied |
| Bending – transversal | satisfied |
| Shear | satisfied |
| Piles | satisfied |

The CFRP lamella are only necessary in the area of the midfield, however in order to secure a sufficient anchoring length the lamella will be applied over half the span, compare the following sketch.


*Figure 4.6: Areas, in which CFRP lamellas are applied, are marked in orange*

In order to evaluate the rehabilitation measure the necessary volume of CFRP has to be computed. It is assumed that the required area of CFRP in every section is the average of section 2 and section 4 [300 mm² per beam]. This assumption is on the safe side, as both analysed sections are the most unfavourable sections.

$$V_{nec} = 4 \times 300\, mm^2 \times \frac{319.75}{2} = 0.192\, m^3 \tag{15}$$

### 4.2.3 Effect if the intervention with CFRP on the durability

The intervention with CFRP has no effects directly on the durability of the existing structure as the zones exposed to the most severe mechanical and environmental effects, the top surface of the bridge deck and the curbs, are not protected. In this case it could be possible that a further intervention (e.g. reducing the permeability of the deck through waterproofing or a repassivation of steel rebars) to reach a total life time of 100 years.

### 4.2.4 Implementation

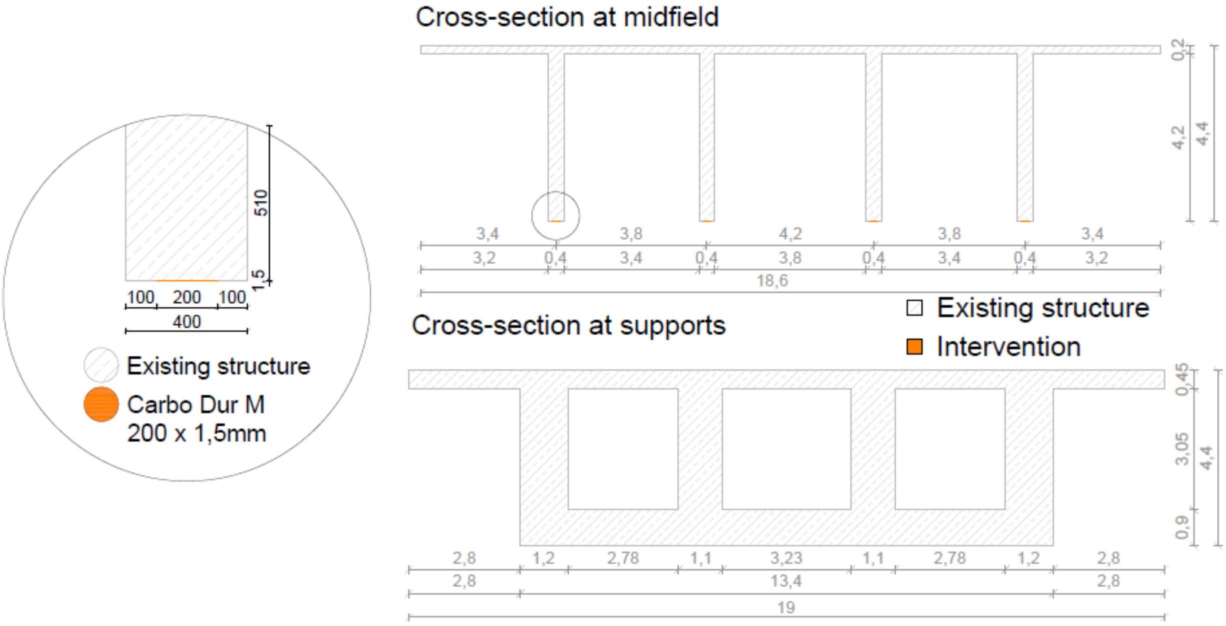



*Figure 4.7: Implementation Plan CFRP [m]; Detail [mm]*

## 4.3 Intervention by decreasing the acting loads

The degree of compliance $n$ is dependent on two factors, see equation (1). Thus, the degree of compliance can be increased in two ways: either by increasing the resistance side or by decreasing the actions, which is regarded in this concept.

In the case of a bridge the main contributing loads are the constant loads (mainly the self-weight) and the traffic loads, which in total have the same order of magnitude as the constant load. The self-weight itself cannot be influenced. However one possibility is to measure the exact geometry of the structure. Through this the partial safety factor for constant loads can be reduced from 1.35 to 1.20, thus reducing the design constant load by 11%. Both the German and the Swiss standard give this value for the reduction of the partial safety factor for constant loads.

The second main contributor to the total load, the traffic loads, can be influenced to a certain degree. According to the German guideline for traffic load models for existing road bridges (Bast 2007) compensation measures can be taken if the bridge does not reach the structural safety. Such compensation measures could be banning the overtaking of trucks, a distance constraint in moving traffic or a limitation of heavy goods vehicles. However compensation measures should only be taken under exceptional circumstances. In all other cases the bridge has to satisfy the structural safety using the traffic load model according to DIN FB 101. Here, the Swiss and the German standard are not conform, as they use very different adjustment factors for the traffic loads on existing bridges, as seen in section 3.7.5 on page 18.

The adjustments factors used by the DIN FB 101 for the tandem loads (TS) are 1.33 times as high and for the universal distributed loads (UDL) twice as high as according to SIA 269-1. Due to this difference the structural safety is satisfied when using the adjustment factors according to SIA 269-1 but not when using DIN FB 101.

The values for the adjustment factor are different for the two countries, even though the current SIA 269-1 and the "*Nachrechnungsrichtlinie*" were published in the same year (2011). One reason for this difference could be the disparity between the traffic in Germany and Switzerland. Germany has a higher traffic volume than Switzerland and thus the guideline for Germany probably considers more traffic, higher congestion time and higher truck loads. This assumption is strengthened by the fact that the Swiss standard for new structures (SIA 261 2014) uses much lower adjustment factors ($\alpha=0.9$, compare SIA 261 10.3.2) than the German standard for new structures EN DIN 1991-2/NA were values over 1 are considered (see section 2.6). On-site measurements in Germany have shown that any older load models (with lower adjustment factors) than the load model according to DIN FB 101 are not sufficient (BMVBS 2013). On the other hand measurements and simulations in Switzerland have shown that the low adjustment factors are sufficient (Meystre & Hirt 2006; SIA 291/1 2011). This shows that in order to compute realistic traffic loads, which secure structural safety but are not overly conservative, measurements would be necessary as traffic loads can differ. Additionally, it demonstrates that the reduction of the adjustment factors from the DIN FB 101 is presumably not feasible.

# 5 Sustainability assessment of the intervention concepts

In order to achieve a holistic approach to the sustainability of a structure, not only the economic and ecological aspects but also the social impact have to be considered over the structure's full life time. These three elements are the pillars of sustainability. Additionally, a technical aspect has to be considered. The assessment is complex as all four main aspects are interlinked, as pictured in Figure 5.1. But whilst the sustainability assessment for buildings is already defined and regulated, the sustainability assessment of infrastructures still incorporates various difficulties. Firstly, no integrated rating system exists, which considers all aspects of sustainability or all different tasks (from the design, to the construction and the maintenance to the demolishment of a structure). Secondly, the sustainability study cannot only be realised for the bridge itself but for the whole road network, thus making the sustainability assessment dependent on the complete road network (de Larderel 2006; Graubner et al. 2011; Pfaffinger et al. 2009).

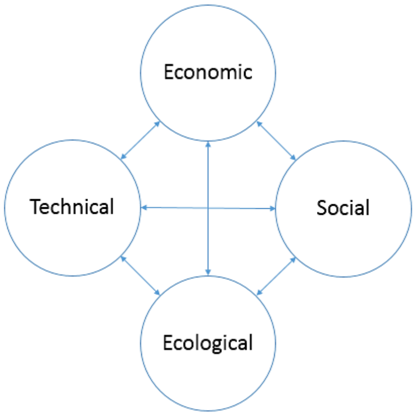



*Figure 5.1: The main aspect of the sustainability assessment and their connections*

The sustainability assessment is more easily interpretable through the comparison of two or more concepts. In the following section, only the two strengthening intervention methods (UHPFRC/CFRP) are compared.

As the sustainability assessment is regarded over the full life time of a structure, the concept of life cycle will be explained in the following, followed by the description of the four different aspects of sustainable infrastructure. As the available information is insufficient for a holistic assessment only a few facets of the two strengthening interventions (UHPFRC/CFRP) will be analysed exemplarily. Nevertheless, the following chapter explains all relevant aspects of the sustainability assessment of bridges and could be used as a manual.

## 5.1 Life cycle

The concept of the life cycle is the basis of every sustainability assessment. According to ISO 14040:2006 a life cycle is defined as "consecutive and interlinked stages of a product system, from raw material acquisition or generation from natural resources to final disposal". Considering all life cycle stages prevents the transfer of problems and assists in detecting the key contributing factor instead of addressing insignificant problems. The main stages of the life cycle of any structure are the construction phase, the operation and maintenance phase, possibly a intervention phase and at the end of life a demolition, as depicted in Figure 5.2 (Kovacic 2014).

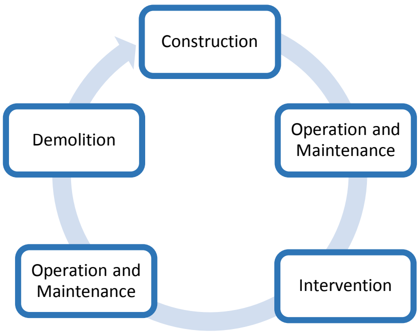



*Figure 5.2: The main phases of the life cycle of a structure*

For the sustainability assessment of the strengthening concepts the life cycle should be considered beginning with the intervention phase, as the sustainability can only be influence form this point forward. Within the three stages intervention, operation and maintenance and demolition various elements have to be considered. The following figure lists the majority of these elements.

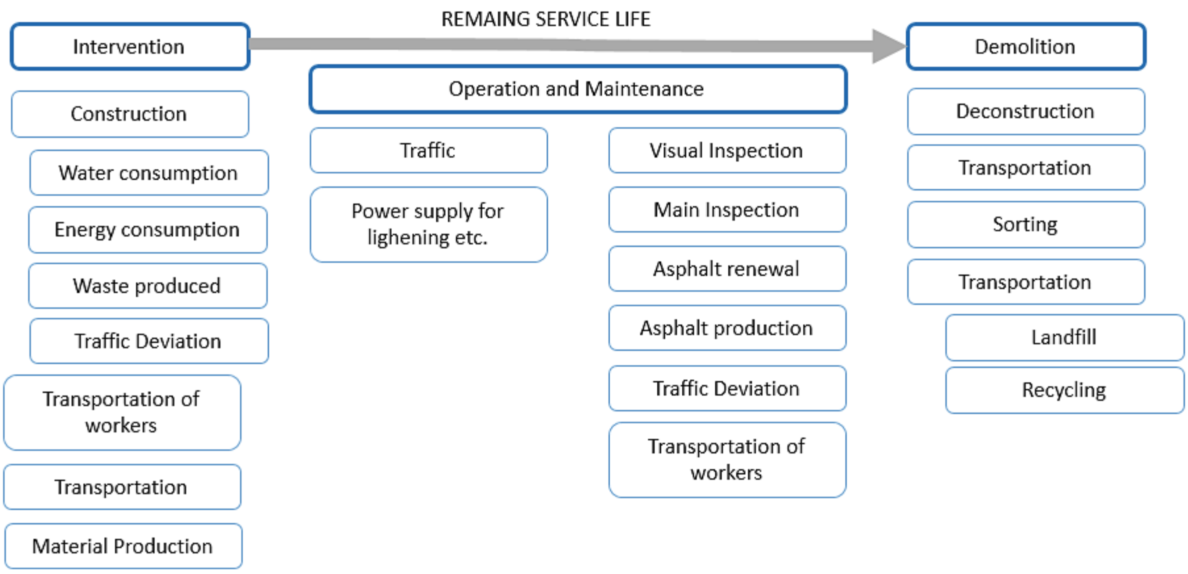



*Figure 5.3: Main elements within the regarded life cycle stages (Dequidt 2012; Habert et al. 2013)*

## 5.2 Economic Aspects

The economic aspect should consider all direct and indirect costs. In general, to increase the transparency of the indirect costs (external costs) and the direct costs (life cycle costs), these are computed separately (Haardt & Schmellekamp 2011).

### 5.2.1 LCC

The direct costs over the life time can be analysed with the life cycle cost (LCC). The main cost components for a structure are the construction, operation and maintenance costs and the demolition costs. The main idea is to compare different investment scenarios to find an optimal solution over the complete life cycle. Reducing the LCC will often be contrary to optimising the two other pillars of sustainability. Thus the LCC should always be considered only as part of the assessment (Kovacic 2014; ÖNORM B 1801-4 2014). In the case of the two strengthening methods both materials (UHPFRC/CFRP) relatively expensive. However the application of UHPFRC does not only secure the structural safety but also increases the durability. Studies have shown that the construction costs of UHPFRC are often similar to conventional methods. However as UHPFRC also increases the durability the overall life cycle cost could be minimised in comparison to the application CFRP (Brühwiler & Denarié 2013).

### 5.2.2 External Costs

Contrarily to buildings, which have nearly no influence on their surroundings, external costs play a vital role in the total costs of infrastructure. The external cost are created through traffic disruptions due to any kind of construction measures. Traffic disruptions lead to additional expenses due to for example loss of time, increased consumption of fuel or an increased risk of accidents. Already small disorders in the network can lead to large congestions and obstructions of traffic (Haardt & Schmellekamp 2011). Thus, the external costs are in this case the most difficult costs to compute. Additionally, little data is available as this is a delicate topic for the owners of road networks. If data on the additional expenses due to traffic disruptions were published, road users might start asking for a refund for their additional expense.

Considering the external costs, the advantage of UHPFRC in comparison to CFRP is that intervention measures could be minimised and subsequently traffic disruption and user costs as well (Brühwiler & Denarié 2013). However, a benefit of the intervention with CFRP would be that the material is applied only at the bottom side of the superstructure of the bridge and thus traffic disruptions during the construction and their implications on the bridge are prevented.

### 5.2.3 Proportionality of maintenance interventions

The proportionality of maintenance interventions described in the SIA 269 is a method, which, in the first instance, measures the economic efficiency of interventions. Thus, for this method only the economic and technical aspects are regarded as a first step. The proportionality of maintenance interventions is based on the comparison of the costs and benefits of maintenance measures with the goal of efficient use of resources. For the cost the direct (LCC) and indirect costs (external costs) have to be considered. For the benefits the reduction of risks, the increase in maintenance value and reliability have to be regarded over the remaining life time. If the maintenance intervention is disproportionate the concept has to be revised or the usage agreement has to be adapted to the circumstances. In case that the maintenance intervention is proportionate it should be implemented (SIA 269 2007).

The SIA 269 gives an equation to estimate the proportionality of safety maintenance measures. The efficiency of the measures ($EF_M$) is given as the ratio of risk reduction due to the maintenance measures ($\Delta R_M$) and the costs ($SC_M$), compare the following equation. $\Delta R_M$ can be determined with the assumption of 3 – 10 million CHF per saved human life. Both the $\Delta R_M$ and the $SC_M$ are formulated as a discounted, annual monetary values over the remaining life time. For the discounting a rate of 2% can be assumed (Brühwiler 2014; SIA 269 2007).

$$EF_M = \frac{\Delta R_M}{SC_M} \geq 1 => Maintenance\ intervention\ proportionate \tag{16}$$

In case the efficiency of the measures is below 1 considering the following aspects could lead to a different evaluation: the security requirements of the individual and society, the availability of a building, the extent of damage to person, property or environment and the preservation of the cultural value (SIA 269 2007). Thus in a second instance a full sustainability assessment should be implemented, considering not only the economic and technical aspects but also the social and ecological.

## 5.3 Ecological Aspects

To assess the ecological aspects the energy consumption and the environmental impact have to be qualified. For the construction of infrastructure project different methods already exist for example the UVP (Umweltverträglichkeitsprüfung – environmental impact assessment). However a holistic method which includes all aspects transparently and comprehensible is still missing (Graubner et al. 2011). An option for a holistic approach is the Life-Cycle Assessment (LCA) (de Larderel 2006).

The ISO 14040:2006 defines life cycle assessment (LCA) as a "compilation and evaluation of the inputs, outputs and the potential environmental impacts of product system throughout its life cycle". The goal is to gain the know-how to protect the environment. By giving a complete overview over the environmental situation the transferal of problems is avoided. Through this holistic approach unilateral views are avoided and relevant environmental impacts are assessed, thus optimal solutions can be found (de Larderel 2006; ISO 14040 2006).

The LCA is divided into four phases: goal and scope definition, inventory analysis, impact assessment and the interpretation (ISO 14040 2006). In the following these stages will be described.

The goal has to be described precisely in order to give significant results. For a scope definition, the first important step is the definition of the system boundaries. The system has to be regarded with a limited time and space and some elements may have to be excluded. Furthermore, the used data and any assumption made have to be determined. Another important aspect is the selection of the indicators; generally the evaluation of the LCA is computed with various indicators, which are divided into two groups: the indicators for the environmental impact and those for the energy consumption. The indicators for the environmental impact are the global warming potential (GWP), Ozone Depletion Potential (ODP), Photochemical Ozone Creation Potential (POCP), Acid Potential (AP) and the Eutrophication Potential (EP) (ISO 14040 2006).

The second phase according to the EN ISO 14040:2006 the life cycle inventory analysis (LCI) phase "involves the compilation and quantification of inputs and outputs for a product throughout its life cycle". The inputs are the various processes necessary for the fabrication of the product, these processes should be subdivided as far as possible in order to obtain unit processes. Inflows generally are material, resources and energy. The output of the LCI are the environmental impacts and the products of each process (Dequidt 2012; ISO 14044 2006; ISO 14040 2006).

Life cycle impact assessment (LCIA) is the third stage. According to EN SIA 14040:2006, LCIA "aims at understanding and evaluating the magnitude and significance of the potential environmental impacts for a product system throughout the life cycle of the product." In this phase the potential environmental impacts and consumed energy are estimated and weighted (Dequidt 2012; ISO 14044 2006; ISO 14040 2006). In the final phase (life cycle interpretation phase) "the findings of either the inventory analysis or the impact assessment or both are evaluated in relation to the defined goal and scope in order to reach conclusion and recommendations", according to EN SIA 14040:2006.

In the case of the bridge by Taubenstein the goal of the LCA is to compare the different intervention measures in order to find the most sustainable solution. The temporal boundary of the system is from the moment of the intervention (age of the bridge is 52 years) to the remaining life time, depicted in Figure 5.3. However, the main processes for the bridge by Taubenstein listed in Figure 1.3 would for a complete and detailed LCI have to be divided into further subsets. The spatial boundaries of the study are the outer edges of the complete structure (foundations, piles, deck, carriageway - concrete surface and the water proofing). The regarded indicators are: GWP, ODP, AP, EP, POCP, total energy used and fossil energy used.

In the following section, one simple aspect of the LCA on the bridge by Taubenstein will be regarded: material production, namely that of UHPFRC, reinforcing steel, CFRP and, for comparison, conventional concrete will be regarded. The full LCI and LCIA are attached in Appendix D. The following diagram compares the impacts and consumed energy of 1 kg of UHPFRC, reinforcing steel, CFRP and conventional concrete. These results have no significance for the LCA as it has to be analysed over the complete volume. However it shows the environmental properties of the production of the four different products.

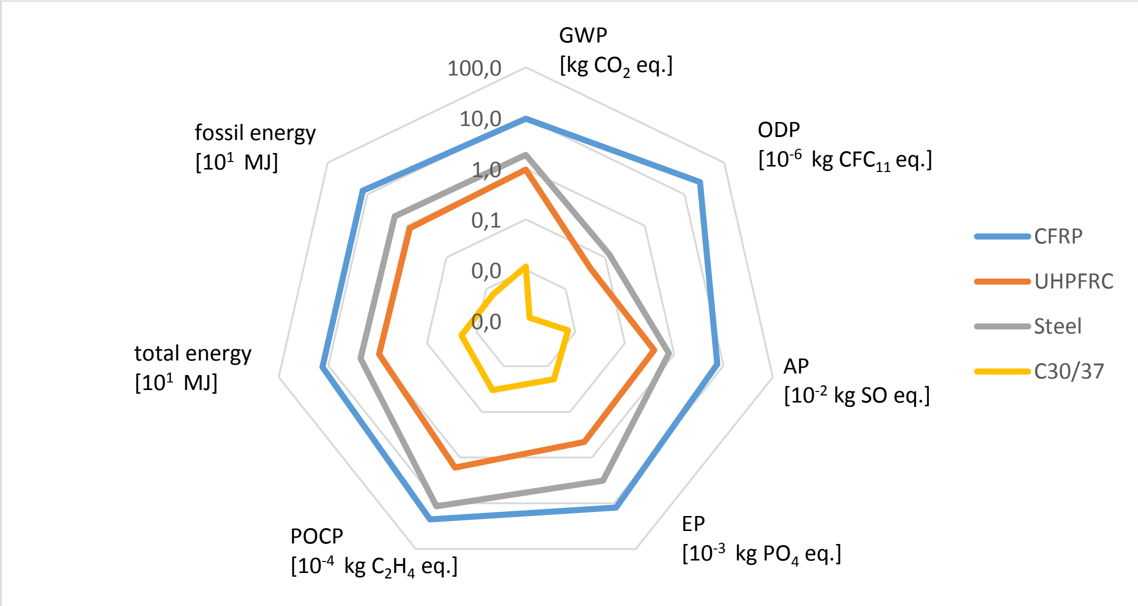



*Figure 5.4: Comparison of the environmental effects and the energy demand of the material production of 1 kg of CFRP, UHPFRC, Steel and conventional concrete (C30/37) on a logarithmic scale*

In order to compare the two different intervention concepts (using R-UHPFRC or CFRP) the necessary volume [m³] of reinforcing steel, UHPFRC and CFRP have to converted into weight (kg). For the reason, that environmental impact and the energy used in the LCIA are given per unit mass.

$$G_{steel} = V_{steel}\ \rho_{steel} = 7.53\ m^3 \times 7850 \frac{kg}{m^3} = 59111\ kg \tag{17}$$

$$G_{UHPFRC} = V_{UHPFRC}\ \gamma_{UHPFRC}\ 101.97 \frac{kg}{kN} = 345.33\ m^3 \times 25 \frac{kN}{m^3} \times 101.97 \frac{kg}{kN} = 880333\ kg \tag{18}$$

$$G_{CFRP} = V_{CFRP}\ \gamma_{CFRP}\ 101.97 \frac{kg}{kN} = 0.192\ m^3 \times 15.7 \frac{kN}{m^3} \times 101.97 \frac{kg}{kN} = 307\ kg \tag{19}$$

The following diagram shows the results of the comparison of the LCA of the material production of the two interventions methods.

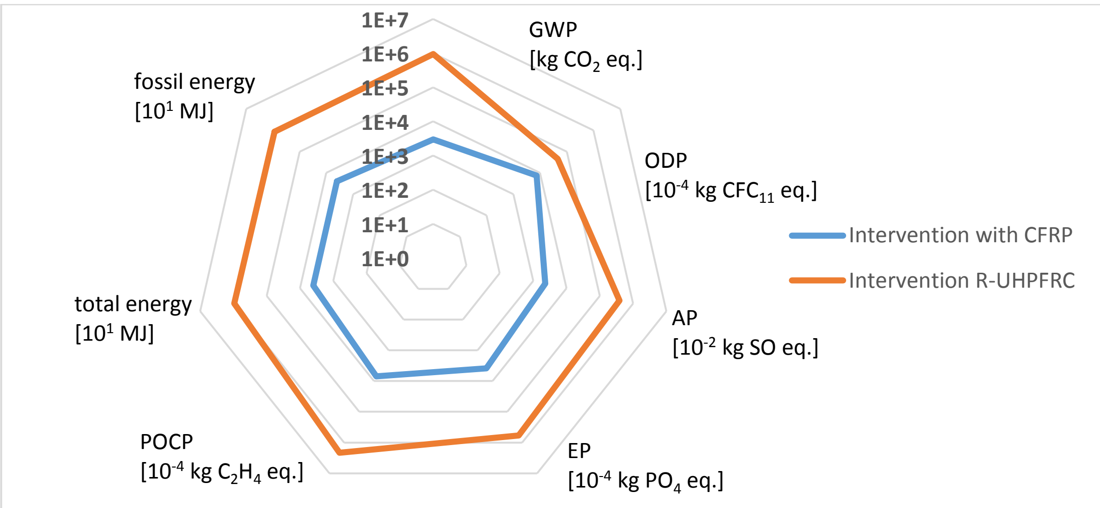



*Figure 5.5: Comparison of the environmental effects and the energy demand of the material production of the intervention with CFRP and the intervention with R-UHPFRC*

The environmental impacts and the energy demand are higher for the intervention with UHPFRC. This is mainly due to the fact that necessary volume for the intervention with UHPFRC is around 1000 times higher than for the intervention with CFRP. However, the intervention with CFRP only influences the structural safety. Whilst it was decided that the layer of UHPFRC is applied over the complete bridge in order to increase its durability. If the LCA were to be developed for the remaining life time, the intervention with UHPFRC would be proven to many advantages. For example, fewer intervention measures are needed and no additional waterproofing is necessary. Additionally, every intervention measure leads to traffic obstructions. These traffic disruptions (Zinke et al. 2014) on a rural highway bridge, such as the bridge by Taubenstein, cause an acidification potential (AP) 400% higher than the impact caused by construction. Thus, the environmental impact and also the energy demand would presumably rise faster for the intervention with CFRP.

## 5.4 Social Aspects

The social aspects include the user satisfaction, traffic safety, operational optimisation and furthermore the preservation of historic and aesthetical buildings. The first three elements are closely linked to the external costs and are equally as complex to quantify.

In order to quantify the historic and aesthetical value of the bridge a similar approach should be considered as with the determination of the maintenance value according to SIA 2015. All parties involved including the state and public should take part in the evaluation of the historic and aesthetical values.

The social aspects of a sustainability assessment are the most complex to evaluate as they are not easily quantifiable and are seldomly objective and thus hard to compare.

## 5.5 Technical Aspects

The technical aspects regard the technical quality according to standards and guidelines and the durability of the structure (Haardt & Schmellekamp 2011). Both technical facets are analysed in the Chapter Concepts of Intervention. With both strengthening intervention concepts the structural safety is reached. Figure 5.6 shows the increase of the degree of compliance $n$ with the intervention methods and the degree of compliance of the existing structure prior to the intervention.

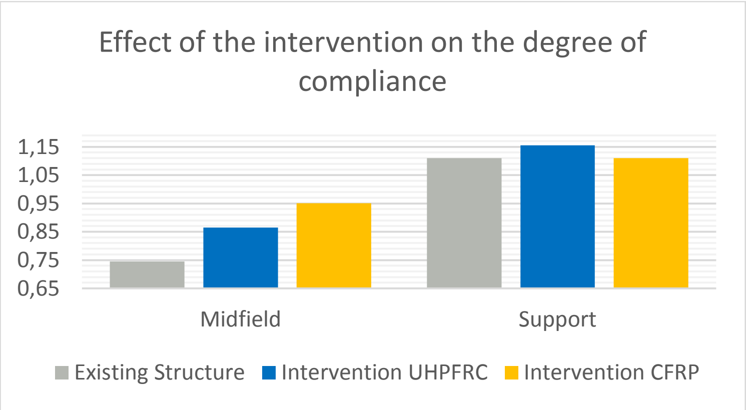



*Figure 5.6: Effects of the intervention on the degree of compliance n regarding the cross-section in midfield and at the supports (most deceive cross-sections)*

In case of the intervention with UHPFRC the durability of the structure is also increased, whereas with the intervention with CFRP only a minimal or even null improvement can be expected. Thus, regarding durability, the intervention with UHPFRC would deliver better results as the risk of deterioration of the concrete and corrosion of the steel rebars is much lower.

# 6 Conclusion

In the following paragraphs, the conclusions drawn from the work above are summarised.

The rehabilitation and maintenance of such a vast amount of existing structures is a relatively new problem and, therefore, less guidelines and standards exist for existing structures than for designing new structures. Switzerland took a leading role by publishing a series of standards on the topic of "Maintenance of Structures" SIA 269. However, the other European countries still miss an encompassing standard. Each country has a different approach with most referring to the older guidelines (e.g. Germany and Austria). The aim should be to design a Eurocode for existing structures considering national differences (for example different traffic volume) with the national appendices.

The examination shows that the existing bridge has to be performed at various levels in high detail. For the detailed examination a non-linear analysis is performed considering the plasticity, cracking behaviour and the history of the structure. The comparison of the general and the detailed examination clearly displays the advantages of analysing the structure in higher detail, as the results of the detailed examination give more realistic results.

Additionally to a detailed computational analysis, an on-site condition survey and evaluation should be performed beforehand. Through these the actual dimensions as well as the exact material parameters and the crack evolution on the structure can be established. With this information a model based completely on the existing structure can be designed.

In general the goal should be to preserve existing structures. This solution is in most cases environmentally, economically and socially friendlier, as the construction of a new bridge would consume enormous amount of energy, material and monies and cause traffic disruptions. The preliminary evidence of this work shows that the application of UHPFRC has many benefits. It's extremely low permeability as wells as its excellent mechanical properties make the material particularly suitable for the local "hardening" of reinforced concrete structures in critical zones, which are exposed to an aggressive environment and considerable loads. UHPFRC structures promise to be a long-term durability thus avoiding frequent maintenance works during the use phase. However, more work and data would be needed for a complete overview and, ideally, a full sustainability assessment of the intervention method should be computed.

The sustainability assessment is good tool for a holistic and integrated evaluation of different scenarios, however, for existing structures, mainly existing bridges, little data is available. As the assessment has not been standardised, the comparison between different projects is complicated. Again, a Europe-wide standardised guideline would enable the development of better benchmarks and simplify comparisons.

In the presented case study, the bridge by Taubenstein, it is shown that an intervention is necessary and more economically, socially and environmentally sustainable, with both strengthening intervention concepts (UHPFRC/CFRP) being a solution. Ideally, both methods should be combined in order to benefit from the advantages of both materials thus reducing cost, environmental impact and further traffic disruption due to maintenance measures.

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# 8 Appendices

## Appendix A: Dimensions of the bridge

The following table gives the exact dimensions of 94 cross-section along the length of the bridge. This data was used to compute the exact self-weight of the bridge. The section highlighted in blue are the four different sections, which were regarded in detail.

| long. coordinate | total height | height compr. plate | height deck | distance webs | width outer web | width inner web |
|---|---|---|---|---|---|---|
| x [m] | H [m] | $h_C$ [mm] | $h_D$ [mm] | A [m] | $b_{Wo}$ [mm] | $b_{Wi}$ [mm] |
| 0.00 | 2.10 | 0 | 400 | 3.80 | 600 | 600 |
| 1.40 | 2.13 | 0 | 200 | 3.80 | 507 | 507 |
| 3.00 | 2.16 | 0 | 200 | 3.80 | 400 | 400 |
| 5.34 | 2.20 | 0 | 200 | 3.80 | 400 | 400 |
| 10.68 | 2.30 | 0 | 200 | 3.80 | 400 | 400 |
| 16.03 | 2.40 | 0 | 200 | 3.80 | 400 | 400 |
| 21.37 | 2.50 | 0 | 200 | 3.80 | 400 | 400 |
| 24.05 | 2.55 | 0 | 200 | 3.80 | 400 | 400 |
| 26.70 | 2.60 | 160 | 200 | 3.80 | 466 | 466 |
| 27.24 | 2.61 | 184 | 200 | 3.80 | 450 | 480 |
| 30.45 | 2.67 | 328 | 200 | 3.80 | 560 | 560 |
| 32.05 | 2.70 | 400 | 400 | 3.80 | 900 | 800 |
| 33.65 | 2.73 | 328 | 200 | 3.80 | 840 | 752 |
| 37.40 | 2.81 | 160 | 200 | 3.80 | 700 | 640 |
| 40.08 | 2.86 | 0 | 200 | 3.80 | 601 | 560 |
| 40.97 | 2.87 | 0 | 200 | 3.80 | 567 | 534 |
| 45.45 | 2.96 | 0 | 200 | 3.80 | 400 | 400 |
| 49.88 | 3.04 | 0 | 200 | 3.80 | 400 | 400 |
| 58.80 | 3.21 | 0 | 200 | 3.80 | 400 | 400 |
| 67.72 | 3.38 | 0 | 200 | 3.80 | 400 | 400 |
| 72.15 | 3.46 | 0 | 200 | 3.80 | 400 | 400 |
| 74.55 | 3.51 | 0 | 200 | 3.80 | 700 | 700 |
| 76.63 | 3.55 | 0 | 200 | 3.80 | 612 | 700 |
| 77.53 | 3.56 | 0 | 200 | 3.80 | 574 | 700 |
| 78.35 | 3.58 | 0 | 200 | 3.80 | 539 | 700 |
| 80.20 | 3.61 | 0 | 200 | 3.80 | 582 | 700 |
| 85.00 | 3.68 | 160 | 200 | 3.80 | 665 | 700 |
| 85.55 | 3.71 | 330 | 200 | 3.80 | 700 | 700 |
| 87.10 | 3.74 | 400 | 400 | 3.80 | 637 | 558 |
| 91.12 | 3.82 | 350 | 200 | 3.80 | 474 | 450 |
| 91.73 | 3.83 | 220 | 200 | 3.80 | 449 | 433 |
| 92.95 | 3.85 | 199 | 200 | 3.80 | 400 | 400 |
| 97.92 | 3.94 | 160 | 200 | 3.80 | 400 | 400 |
| 104.10 | 4.06 | 160 | 200 | 3.80 | 400 | 400 |
| 110.28 | 4.17 | 160 | 200 | 3.80 | 400 | 400 |
| 115.25 | 4.26 | 407 | 200 | 3.80 | 400 | 1180 |
| 116.47 | 4.29 | 605 | 200 | 3.80 | 528 | 1101 |

| x [m] | H [m] | $h_C$ [mm] | $h_D$ [mm] | A [m] | $b_{Wo}$ [mm] | $b_{Wi}$ [mm] |
|---:|---:|---:|---:|---:|---:|---:|
| 117.09 | 4.30 | 653 | 200 | 3.80 | 593 | 1061 |
| 120.95 | 4.37 | 678 | 200 | 3.80 | 1001 | 810 |
| 122.65 | 4.40 | 830 | 200 | 3.80 | 1200 | 1100 |
| 124.35 | 4.40 | 900 | 400 | 3.80 | 1132 | 1051 |
| 131.99 | 4.40 | 810 | 200 | 3.80 | 827 | 773 |
| 136.65 | 4.40 | 407 | 200 | 3.80 | 640 | 610 |
| 141.32 | 4.40 | 160 | 200 | 3.80 | 453 | 447 |
| 142.65 | 4.40 | 0 | 200 | 3.80 | 400 | 400 |
| 150.66 | 4.40 | 0 | 200 | 3.80 | 400 | 400 |
| 159.99 | 4.40 | 0 | 200 | 3.80 | 400 | 400 |
| 169.33 | 4.40 | 0 | 200 | 3.80 | 400 | 400 |
| 178.66 | 4.40 | 0 | 200 | 3.80 | 400 | 400 |
| 188.00 | 4.40 | 0 | 200 | 3.80 | 400 | 400 |
| 194.00 | 4.40 | 0 | 200 | 3.80 | 400 | 400 |
| 196.00 | 4.40 | 0 | 200 | 3.80 | 400 | 400 |
| 197.34 | 4.40 | 0 | 200 | 3.81 | 453 | 447 |
| 202.00 | 4.40 | 160 | 200 | 3.81 | 640 | 610 |
| 206.67 | 4.40 | 407 | 200 | 3.83 | 827 | 773 |
| 214.30 | 4.40 | 810 | 200 | 3.91 | 1132 | 1041 |
| 216.00 | 4.40 | 900 | 450 | 3.93 | 1200 | 1100 |
| 217.70 | 4.36 | 837 | 210 | 3.96 | 1003 | 818 |
| 222.00 | 4.27 | 678 | 210 | 4.02 | 506 | 1116 |
| 222.67 | 4.25 | 653 | 210 | 4.04 | 429 | 1162 |
| 222.92 | 4.25 | 644 | 210 | 4.04 | 400 | 1180 |
| 229.33 | 4.11 | 407 | 210 | 4.16 | 400 | 400 |
| 236.00 | 3.96 | 160 | 210 | 4.31 | 400 | 400 |
| 242.67 | 3.81 | 160 | 210 | 4.48 | 400 | 400 |
| 248.00 | 3.70 | 160 | 210 | 4.64 | 400 | 400 |
| 249.33 | 3.67 | 200 | 210 | 4.67 | 450 | 433 |
| 250.00 | 3.65 | 220 | 210 | 4.69 | 475 | 450 |
| 254.45 | 3.55 | 354 | 210 | 4.84 | 642 | 561 |
| 256.00 | 3.52 | 400 | 400 | 4.89 | 700 | 600 |
| 257.55 | 3.49 | 307 | 230 | 4.93 | 642 | 561 |
| 260.00 | 3.43 | 160 | 230 | 5.01 | 550 | 500 |
| 262.00 | 3.39 | 0 | 230 | 5.07 | 475 | 450 |
| 262.67 | 3.37 | 0 | 230 | 5.09 | 450 | 433 |
| 264.00 | 3.34 | 0 | 230 | 5.13 | 400 | 400 |
| 269.33 | 3.22 | 0 | 230 | 5.29 | 400 | 400 |
| 276.00 | 3.07 | 0 | 230 | 5.49 | 400 | 400 |
| 282.67 | 2.93 | 0 | 230 | 5.67 | 400 | 400 |
| 288.00 | 2.81 | 0 | 230 | 5.81 | 400 | 400 |
| 289.33 | 2.78 | 0 | 230 | 5.85 | 467 | 450 |
| 290.00 | 2.76 | 0 | 230 | 5.88 | 500 | 475 |
| 292.00 | 2.72 | 160 | 230 | 5.91 | 600 | 550 |
| 294.10 | 2.67 | 283 | 230 | 5.99 | 705 | 629 |

| x [m] | H [m] | $h_C$ [mm] | $h_D$ [mm] | A [m] | $b_{Wo}$ [mm] | $b_{Wi}$ [mm] |
|---|---|---|---|---|---|---|
| 296.00 | 2.63 | 400 | 450 | 6.01 | 800 | 700 |
| 297.90 | 2.59 | 286 | 250 | 6.05 | 537 | 537 |
| 299.56 | 2.55 | 186 | 250 | 6.08 | 481 | 481 |
| 300.00 | 2.54 | 160 | 250 | 6.09 | 467 | 467 |
| 302.00 | 2.50 | 0 | 250 | 6.14 | 400 | 400 |
| 303.92 | 2.45 | 0 | 250 | 6.17 | 400 | 400 |
| 307.88 | 2.36 | 0 | 250 | 6.24 | 400 | 400 |
| 311.83 | 2.28 | 0 | 250 | 6.31 | 400 | 400 |
| 315.79 | 2.18 | 0 | 250 | 6.36 | 400 | 400 |
| 316.75 | 2.17 | 0 | 250 | 6.38 | 400 | 400 |
| 318.05 | 2.14 | 0 | 250 | 6.39 | 508 | 487 |
| 319.75 | 2.10 | 0 | 450 | 6.41 | 650 | 600 |

## Appendix B: General examination of the beam model

The results of the general examination on the beam model are only used for as first approach and in order to verify the results of the more complex shell model.

Only the bending of the deck is analysed. To simplify the normal force, the shear force in y-direction, the moment in z-direction and the torsion are neglected. The verifications will be initially computed by hand and in a second step computed with FAGUS 7.0 for comparison.

The following figures display the distribution of the bending moment $M_y$ once due to the self-weight and once due to the traffic load tandem system. These distributions show the significance of the four selected cross-sections.

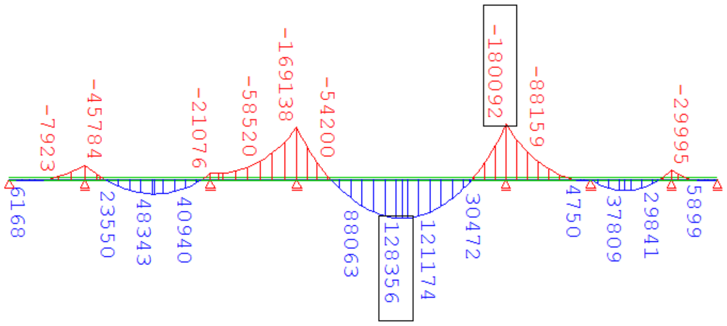


*Figure 8.1: Internal force distribution of the moment $M_y$ due to self-weight on the beam model [kNm]*

The distribution of the moment due to self-weight on the shell model is different than the beam model. This is due to two factors. Firstly and mainly that the carriageway slab is modelled as an orthotropic plate thus distributing the forces differently. Secondly the stiffness of so a compression slab was added in cross-sections of some areas (supports) in this model, which increases the stiffness of the cross-sections in these areas.

The values for each section and every relevant influence are summarised in the following tables for the bending moment $M_y$ and the shear force $V_z$.

*Table 8.1: Bending moment $M_y$ due to the different loads for the sections 1-4.*

| Section | My - G1 [kNm] | My - G2 [kNm] | My – TS max [kNm] | My – TS min [kNm] | My – ULD max [kNm] | My – UDL min [kNm] |
|---|---|---|---|---|---|---|
| 1 | -180092 | -68128 | +23668 | -33772 | +96749 | -145862 |
| 2 | +128356 | +51828 | +30326 | -15751 | +151498 | -115200 |
| 3 | -169138 | -63144 | +24000 | -34787 | +126827 | -174336 |
| 4 | +48343 | +21369 | +23450 | -18116 | +85919 | -59958 |

*Table 8.2: Shear force $V_z$ due to the different loads for the sections 1-4.*

| Section | Vz - G1 [kN] | Vz - G2 [kN] | Vz – TS max [kN] | Vz – TS min [kN] | Vz – ULD max [kN] | Vz – ULD min [kN] |
|---|---|---|---|---|---|---|
| 1 | -15556 | -5081 | +1521 | -1390 | +2877 | -2993 |
| 2 | 0 | 0 | +805 | -751 | +1517 | -1633 |
| 3 | +15322 | +4972 | +1406 | -1532 | +3164 | -3022 |
| 4 | +1244 | +544 | +863 | -879 | +1208 | -1116 |

The table lists the bending moment at each sections firstly according to "*Nachrechnungsrichtlinie*" and secondly according to SIA 269, see section 3.7.5 on page 18. To simplify the recalculation the resisting bending moment is computed for one of the four T-beams. These values are compared with the bending moment of one T-beam, therefore the bending moments have to be divided by 4, see Table 8.3.

*Table 8.3: Bending moments of each section according to DIN, SIA and for one beam*

| Section | $M_y$ - DIN [kNm] | $M_y$ – SIA [kNm] | $M_y$ for 1 beam – DIN [kNm] | $M_y$ for 1 beam –SIA [kNm] |
|---|---|---|---|---|
| 1 | -567315 | -486480 | -141829 | -121620 |
| 2 | 488957 | 407136 | 122239 | 101784 |
| 3 | -592423 | -498318 | -148106 | -124579 |
| 4 | 247708 | 198492 | 61927 | 49623 |

The following equations (DIN EN 1992-1-1 2011) are used to calculate the resistance moment for each beam in the four cross-sections.

$$x_{B,lim,p} = \frac{2.8 * dp}{3.5 + \varepsilon_{py}} \tag{20}$$

$$x_{B,p0} = \frac{Fp}{b_{eff} * f_{cd}} \tag{21}$$

$$Fcf = hf * (beff - bw) * fcd \tag{22}$$

$$x_{B,p} = \frac{Fp - Fcf}{b_w * f_{cd}} \tag{23}$$

$$x_{B,p} \leq hf => Case\ A; hf \leq x_{B,p} \leq x_{B,lim,p} => Case\ B;\ x_{B,p} \geq x_{B,lim,p} => Case\ C \tag{24}$$

$$M_{Rd} = Fcf * \left(dp - \frac{hf}{2}\right) + x_{B,p} * bw * fcd * (dp - \frac{x_{B,p}}{2}) \tag{25}$$

*Table 8.4: Computation of the resisting moment due to the pre-stressing*

| Sec. | $d_p$ [m] | $b_{eff}$ [m] | $F_p$ [MN] | $x_{b,lim,p}$ [m] | $x_{Bp0}$ [m] | $F_{cf}$ [MN] | $x_{Bp}$ [m] | Case | $M_{rd}$ [MNm] |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 4 | 1.53 | 32.4 | 3.18 | 1.592 | 4.549 | 1.820948 | B | 102.195 |
| 2 | 4 | 2.03 | 25.92 | 3.18 | 0.960 | 4.336 | 4.05718 | C | 59.461 |
| 3 | 4 | 1.53 | 32.4 | 3.18 | 1.592 | 4.195 | 1.844078 | B | 101.853 |
| 4 | 3 | 2.03 | 16.2 | 2.39 | 0.600 | 4.336 | 2.230113 | B | 34.937 |

For all sections the resisting moment only due to the concrete and the pre-stressing is not enough, therefore additional reinforcement is necessary (DIN EN 1992-1-1 2011).

$$x_{B,lim} = \frac{560 * d}{700 + fyd} \tag{26}$$

$$Mcf,s1 = Fcf * (d - \frac{hf}{2}) \tag{27}$$

$$x_B = d - \sqrt{d^2 - \frac{2(Med - Mcf,s1)}{bw * fcd}} \tag{28}$$

$$x_B \leq hf => Case\ A; hf \leq\ x_B \leq x_B => Case\ B;\ x_B \geq x_B => Case\ C; \tag{29}$$

$$\sqrt{<0} => Case\ C \tag{30}$$

$$As2 = \frac{Med - Mrd}{fyd * z} \tag{31}$$

$$As1 = \frac{Fcf + x_{B,lim} * bw * fcd + As2 * 478{,}3 - Fp}{478.3} \tag{32}$$

*Table 8.5: Computation of the necessary reinforcement and the degree of compliance n with minimal reinforcement – DIN*

| Sec. | $x_{b,lim}$ [m] | $M_{cf,si}$ [MNm] | d [m] | $x_B$ [m] | $x_B$=$x_{B,lim}$ [m] | $A_{s1}$ [cm²] | $A_{s2}$ [cm²] | $A_{s,min}$ [cm²] | n [-] | n [-] FAGUS |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 1.9 | 1.9 | 4.3 | 2.9 | 1.902 | 232.7 | 207.15 | 253.4 | 0.8 | 1.1 |
| 2 | 1.9 | 1.9 | 4.3 | √<0 | 1.902 | 88.9 | 328.13 | 49.6 | 0.3 | 0.6 |
| 3 | 1.9 | 1.9 | 4.3 | 3.3 | 1.902 | 259.9 | 241.75 | 222.1 | 0.9 | 1.1 |
| 4 | 1.4 | 1.4 | 3.1 | √<0 | 1.426 | 98.6 | 188.09 | 38.2 | 0.3 | 0.7 |

*Table 8.6: Computation of the necessary reinforcement and the degree of compliance n with minimal reinforcement – SIA*

| Sec. | $x_{b,lim}$ [m] | $M_{cf,si}$ [MNm] | d [m] | $x_B$ [m] | $x_B$=$x_{B,lim}$ [m] | $A_{s1}$ [cm²] | $A_{s2}$ [cm²] | $A_{s,min}$ [cm²] | n [-] | n [-] FAGUS |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 1.9 | 1.9 | 4.3 | 2.9 | 1.902 | 167.3 | 141.7 | 253.4 | 2.2 | 1.3 |
| 2 | 1.9 | 1.9 | 4.3 | √<0 | 1.902 | -33.9 | 205.8 | 49.6 | 0.5 | 0.8 |
| 3 | 1.9 | 1.9 | 4.3 | 3.3 | 1.902 | 128.7 | 110.5 | 222.1 | 1.7 | 1.3 |
| 4 | 1.4 | 1.4 | 3.1 | √<0 | 1.426 | 9.6 | 99.0 | 38.2 | 0.6 | 0.9 |

The table lists the shear forces at each sections firstly according to "*Nachrechnungsrichtlinie*" and according to SIA 269. As with the verification of the bending moment the recalculation resisting shear strength of each beam is computed and are therefore also compared with the bending moment of each beam.

*Table 8.7: Shear force of each section according to DIN, SIA and for one beam*

| Section | $V_z$ – EC [kN] | $V_z$ – SIA [kN] | $V_z$ – EC - per beam [kN] | $V_z$ – SIA- per beam [kN] |
|---|---|---|---|---|
| 1 | -31339 | -29367 | -7835 | -7342 |
| 2 | 3483 | 2438 | 871 | 610 |
| 3 | 31208 | 29151 | 7802 | 7288 |
| 4 | 5252 | 4320 | 1313 | 1080 |

In the following the equations according to DIN EN 1992 are given to calculate the resisting shear strength of the cross-section without shear reinforcement.

$$V'_{Rd,c} = [C_{Rd,c} * \kappa * (100 * \rho l * fck)^{\frac{1}{3}} + 0{,}12 * \sigma cp] * bw * d \tag{33}$$

$$V'_{Rd,c} = [0{,}1 * (1 + \sqrt{\frac{20}{d}}) * \left(100 * \frac{Asl}{bw * d} * fck\right)^{\frac{1}{3}} + 0{,}12 * \frac{N_{Ed}}{A}] * bw * d \tag{34}$$

$$v_{min} = \left(\frac{\kappa_1}{1.5}\right) * \sqrt{\kappa^3 * fck} \tag{35}$$

$$V_{Rd,c,min} = (v_{min} + 0.12 * \sigma cp) * beff * d \tag{36}$$

$$V_{Rd,c} = \min[V'_{Rd,c};\ V_{Rd,c,min}] \tag{37}$$

As listed in the Table 8.8 the resisting shear strength of the concrete is not sufficient in all four sections, therefore shear reinforcement is necessary. With the following equations the resisting shear strength in the tension strut and a compression strut can be computed.

$$V_{Rd,max} = 1.0 * b_w * z * 0.6 * \left(1 - \frac{f_{ck}}{250}\right) * f_{cd} * \frac{1}{1.667 + 0.6} \tag{38}$$

$$V_{Rd,s} = a_{sw} * z * f_{ywd} * \cot\theta \tag{39}$$

*Table 8.8: Resisting shear strength and degree of compliance n*

| Sec. | $A_{sl}$ [cm2] | $V'_{Rd,c}$ [kN] | $V_{Rd,c,min}$ [kN] | $V_{Rd,c}$ [kN] | $V_{Rd,max}$ [kN] | $a_{sw}$ [cm²/m] | cot θ [-] | $V_{Rd,s}$ [kN] |
|---|---|---|---|---|---|---|---|---|
| 1 | 10.62 | 398.5 | 915.6 | 915.6 | 14414.9 | 53.6 | 0.6 | 16525.7 |
| 2 | 10.62 | 197.1 | 318.5 | 318.5 | 5013.9 | 20.5 | 0.6 | 6258.8 |
| 3 | 10.62 | 398.5 | 915.6 | 915.6 | 14414.9 | 56.3 | 0.6 | 16525.7 |
| 4 | 10.62 | 160.3 | 233.5 | 233.5 | 3614.7 | 20.5 | 0.6 | 4512.2 |

*Table 8.9: Degree of compliance n*

| Sec. | n - DIN [-] | n – DIN FAGUS [-] | n – SIA [-] | n – SIA FAGUS [-] |
|---|---|---|---|---|
| 1 | 2.1 | 0.8 | 2.3 | 0.8 |
| 2 | 7.1 | 4.8 | 10.0 | 7.1 |
| 3 | 2.1 | 0.8 | 2.3 | 0.9 |
| 4 | 3.4 | 2.3 | 4.2 | 2.8 |

The utilisation factors computed with the results of the beam model by hand and with FAGUS 7.0 are computed in a first step however the values are very conservative. The difference between the results of the calculations by hand and the computation with FAGUS 7.0 is due to different approach. In FAGUS 7.0 the resistance was computed for the complete cross-section while in case of the manual calculation the resistance was determined for one T-Beam and then multiplied by four. The results derived with FAGUS 7.0 are more realistic as the complete cross-section and the pre-deformation due to the pre-stress are considered.

# Appendix C: Internal forces and load combinations

In the following the internal forces of the three different load combinations are listed. The internal forces are computed with the program SOFISTIK and the load combinations by hand.

Load combinations for N

| **Load Combinations with α from SIA** | | | |
|---|---|---|---|
| Section | LC 1 | LC 2 | LC 3 |
| | [kN] | [kN] | [kN] |
| 1 | -13375 | -12549 | -6518 |
| 2 | 13074 | 12316 | 6168 |
| 3 | -12446 | -11991 | -5610 |
| 4 | 12549 | -12030 | -5489 |

| **Load Combinations with α from DIN** | | | |
|---|---|---|---|
| Section | LC 1 | LC 2 | LC 3 |
| | [kN] | [kN] | [kN] |
| 1 | -18590 | -15492 | -8703 |
| 2 | 18355 | 15294 | 8383 |
| 3 | -17660 | -14935 | -7793 |
| 4 | 17925 | 15132 | 7779 |

Load combinations for $V_y$

| **Load Combinations with α from SIA** | | | |
|---|---|---|---|
| Section | LC 1 | LC 2 | LC 3 |
| | [kN] | [kN] | [kN] |
| 1 | -6546 | -6111 | -4610 |
| 2 | 2122 | -2171 | -966 |
| 3 | -5283 | -4796 | -2939 |
| 4 | 7660 | 7036 | 5695 |

| **Load Combinations with α from DIN** | | | |
|---|---|---|---|
| Section | LC 1 | LC 2 | LC 3 |
| | [kN] | [kN] | [kN] |
| 1 | -8428 | -7086 | -5485 |
| 2 | 3032 | 2763 | 1456 |
| 3 | -7469 | -6021 | -3930 |
| 4 | 9440 | 7960 | 6520 |

Load combinations for $V_z$

| **Load Combinations with α from SIA** | | | |
|---|---|---|---|
| Section | LC 1 | LC 2 | LC 3 |
| | [kN] | [kN] | [kN] |
| 1 | -29418 | -28190 | -27193 |
| 2 | 1791 | -2171 | -1358 |
| 3 | 29389 | 28106 | 27049 |
| 4 | 3567 | 2858 | 2485 |

| **Load Combinations with α from DIN** | | | |
|---|---|---|---|
| Section | LC 1 | LC 2 | LC 3 |
| | [kN] | [kN] | [kN] |
| 1 | -31373 | -29216 | -28089 |
| 2 | 2559 | 2178 | 1061 |
| 3 | 31430 | 29181 | 27981 |
| 4 | 4566 | 3370 | 2955 |

Load combinations for $M_T$

| **Load Combinations with α from SIA** | | | |
|---|---|---|---|
| Section | LC 1 | LC 2 | LC 3 |
| | [kNm] | [kNm] | [kNm] |
| 1 | -18545 | -15860 | -12677 |
| 2 | 10135 | -7487 | -4779 |
| 3 | -22983 | -20019 | -14084 |
| 4 | 14062 | 10729 | 7414 |

| **Load Combinations with α from DIN** | | | |
|---|---|---|---|
| Section | LC 1 | LC 2 | LC 3 |
| | [kNm] | [kNm] | [kNm] |
| 1 | -24241 | -18832 | -15307 |
| 2 | 14478 | 10292 | 7142 |
| 3 | -29953 | -23910 | -18087 |
| 4 | 20038 | 13829 | 10190 |

## Appendix D: Complete LCI of the material production for four materials

The following table lists the complete life cycle inventory of the production of the materials used for the intervention UHPFRC and CFRP and a conventional building materials – steel and concrete – for a comparison. The last column indicates the source of the information:

a – Database: Idemat (Idemat 2015)

b – Database: Ecoivent (Ecoinvent 2015)

c – Life cycle assessment of ultra high performance concrete structures (Stengel & Schießl 2014)

| | GWP | ODP | AP | EP | POCP | total energy | fossil energy | |
|---|---|---|---|---|---|---|---|---|
| | kg $CO_2$ | kg $CFC_{11}$ | kg $SO_2$ | kg $PO_4$ | kg $C_2H_4$ | MJ | MJ | |
| MATERIAL PRODUCTION | | | | | | | | |
| Carbon fibre (50%) | 1.3E+01 | 4.9E-05 | 1.1E-01 | 1.9E-02 | 3.4E-03 | 1.4E+02 | 1.4E+02 | a |
| Epoxy resin (50%) | 6.7E+00 | 1.8E-09 | 4.0E-02 | 5.8E-03 | 1.2E-03 | 1.2E+02 | 1.2E+02 | a |
| CFRP | 9.6E+00 | 2.5E-05 | 7.5E-02 | 1.3E-02 | 2.3E-03 | 1.3E+02 | 1.3E+02 | |
| Steel production, converter (66%) [kg] | 1.9E+00 | 1.4E-07 | 7.8E-03 | 3.3E-03 | 1.5E-03 | 2.0E+01 | 1.9E+01 | b |
| Steel production, electric (34%) [kg] | 8.0E-01 | 4.1E-08 | 4.1E-03 | 1.6E-03 | 2.0E-04 | 1.2E+01 | 1.0E+01 | b |
| hot rolling, steel (100%) [kg] | 3.1E-01 | 2.2E-08 | 1.3E-03 | 4.5E-04 | 1.5E-04 | 4.6E+00 | 4.1E+00 | b |
| Steel - reinforced | 1.9E+00 | 1.3E-07 | 7.9E-03 | 3.2E-03 | 1.2E-03 | 2.2E+01 | 2.0E+01 | |
| Steel fibres [kg] | 3.4E+00 | 1.9E-07 | 1.8E-02 | 1.4E-03 | 8.2E-04 | 4.1E+01 | 3.9E+01 | c |
| Portland cement [kg] | 8.7E-01 | 2.1E-08 | 1.7E-03 | 4.1E-04 | 4.6E-05 | 3.9E+00 | 3.3E+00 | b |
| Transport [tkm; 0,0044tkm/kg] | 1.7E-02 | 2.8E-08 | 1.0E-03 | 2.0E-04 | 2.8E-05 | 2.8E+00 | 2.7E+00 | b |
| Electricity [kWh;0,0485kWh/kg] | 4.9E-01 | 2.1E-08 | 1.8E-03 | 1.9E-03 | 5.5E-05 | 1.1E+01 | 5.7E+00 | b |
| Cement [kg] | 8.9E-01 | 2.2E-08 | 1.8E-03 | 5.0E-04 | 4.9E-05 | 4.5E+00 | 3.6E+00 | b |
| Microsilicat [kg] | by-product, no negative environmental impact | | | | | | | b |
| Sand | 1.2E-02 | 1.5E-09 | 8.2E-05 | 1.8E-05 | 2.8E-06 | 1.8E-01 | 1.7E-01 | b |
| Super-plasticiser | 1.3E+00 | 2.5E-07 | 7.8E-03 | 1.9E-03 | 4.1E-04 | 3.4E+01 | 3.1E+01 | b |
| Water | 3.9E-04 | 2.7E-11 | 1.8E-06 | 9.1E-07 | 9.7E-08 | 7.1E-03 | 4.4E-03 | b |
| UHPFRC | 9.6E-01 | 4.4E-08 | 4.0E-03 | 4.5E-04 | 1.6E-04 | 9.3E+00 | 8.5E+00 | |
| Cement | 4.6E-03 | 2.2E-10 | 1.9E-05 | 2.9E-06 | 8.2E-07 | 4.8E-02 | 4.2E-02 | b |
| Sand | 1.2E-02 | 1.5E-09 | 8.2E-05 | 1.8E-05 | 2.8E-06 | 1.8E-01 | 1.7E-01 | b |
| Gravel | 1.0E-02 | 5.3E-10 | 6.0E-05 | 2.0E-05 | 3.2E-06 | 1.4E-01 | 2.9E-02 | b |
| Water | 3.9E-04 | 2.7E-11 | 1.8E-06 | 9.1E-07 | 9.7E-08 | 7.1E-03 | 4.4E-03 | b |
| Superplasticser | 1.3E+00 | 2.5E-07 | 7.8E-03 | 1.9E-03 | 4.1E-04 | 3.4E+01 | 3.1E+01 | b |
| C30/37 | 1.2E-02 | 1.2E-09 | 7.1E-05 | 1.9E-05 | 3.3E-06 | 2.0E-01 | 1.3E-01 | |