# Assessment and Maintenance of Bridges – Requirements, Objectives, and Strategies

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# Foreword

Bridges are important elements of our traffic infrastructure—a system, which was built during the past one hundred years with a major increase between the 1960s and 1980s. Hence, an important number of bridges has now reached an age of forty or more years and thorough inspections and assessments are required. In the years of intense construction activities, the expected quality was not always reached and durability aspects were not in the focus of planners and builders. These facts led to a disproportionate number of defects and damages, quite often to a serious extent. This is relevant beyond a pure technical meaning since the costs for maintenance and repair often exceed the respective budget.

Grischa Dette treats the topic of bridge maintenance in a comprehensive approach. He presents a method for a specific and stepwise maintenance planning and by this offers a promising instrument for the support of strategy selections and decision making processes. By taking up ideas from international literature, Mr. Dette develops an independent evaluation framework, which profoundly extends the common economic perspective. In the main part of the thesis, a newly developed Matlab based computer program SAMS (**S**imulation-based **A**ssessment of **M**aintenance **S**trategies) is introduced. For the evaluation of maintenance strategies, Mr. Dette defines so called performance indicators, which comprise the categories structural safety, condition, economy, society & culture, and environment. This procedure is experimentally applied on two case study bridges.

The holistic approach of this thesis is remarkable since it captures bridges not only as technical structures, but also accounts for the many other roles they play in everyday life. With the broad applicability of the approach and its comprehensible presentation the publication at hand will be of interest for a wide readership.

Lucerne (Switzerland), 2016

Prof. Dr. Viktor Sigrist

## Abstract

From an ideal point of view, bridges should be designed, constructed and operated in such a way that they fulfil requirements from the fields of function, economy, environment, society, and culture. The present thesis wants to contribute to this matter by illuminating the manifold requirements and by developing a practicable approach to multi-objective maintenance planning for bridges.

Starting with a literature review and a survey on practical maintenance planning by bridge owning agencies in Hamburg (Germany) and Chennai (India), the investigation reveals that until today practical approaches for the identification of bridge specific requirements as well as for strategy assessment and evaluation are not at hand. Instead of accounting for the whole range of performance aspects, the current investigations of maintenance strategies are commonly confined to functional and economic aspects. In exceptional cases where societal, cultural and also environmental issues are considered, these aspects are usually treated in monetary terms. A common monetary evaluation of the different aspects, however, inevitably causes a distorted perception of reality.

As a solution to this problem, the present thesis develops a practical approach to multi-objective maintenance planning. The aim of this approach is to develop customed strategies in order to fulfil the bridge specific requirements. At its core is the newly invented SAMS program, which allows for simulation based assessment of maintenance strategies. The program uses Monte Carlo simulation technique and thoroughly takes into account the interdependencies between structure, deterioration, and maintenance effects, which occur in the bridge specific context. As a result, it provides value distributions of indicators, which give a description of the strategy-depended bridge performance relating to condition, structural safety, economy, society, and culture. The SAMS program is effectively combined with a specifically developed method for true and undistortive multi-objective strategy evaluation. In addition to this, a questionnaire for identifying bridge specific requirements is applied—a tool newly developed for the survey in Hamburg and Chennai. The experimental application of the approach on two case study bridges shows that suitable strategies for fulfilling the bridge specific requirements can be identified. It is also found that the bridge specific context decisively influences the extent, to which strategies can play out their specific strengths.

# Kurzfassung


Idealerweise sollten Brückenbauwerke so geplant, errichtet und betrieben werden, dass sie ein breites Anforderungsspektrum erfüllen – insbesondere hinsichtlich der Dimensionen Funktionalität, Wirtschaftlichkeit, Umwelt, Gesellschaft und Kultur. Die vorliegende Untersuchung will hierzu einen Beitrag leisten, indem sie die vielfältigen Anforderungen genauer beleuchtet und einen praktikablen Ansatz für eine mehrzielige Instandhaltungsplanung für Brücken entwickelt.

Ausgehend von einer Literaturrecherche und einer Datenerhebung zur Instandhaltungspraxis, die in Kooperation mit Brückenverwaltungen in Hamburg (Deutschland) und Chennai (Indien) durchgeführt wurde, zeigt die Studie, dass bis heute weder für die Identifizierung von brückenspezifischen Anforderungen noch für die Auswirkungsprognose und Bewertung von Instandhaltungsstrategien praktikable Ansätze zur Verfügung stehen. Anstatt das gesamte Leistungsspektrum der Bauwerke zu betrachten, beschränken sich aktuelle Untersuchungen üblicherweise auf funktionale und wirtschaftliche Aspekte. Dort, wo ausnahmsweise auch gesellschaftlich-kulturelle oder auch umweltbezogene Aspekte berücksichtigt werden, erfolgt üblicherweise eine monetäre Transformation und Bewertung. Eine gemeinsame monetäre Bewertung der unterschiedlichen Dimensionen jedoch führt zwangsläufig zu einem verzerrten Abbild der Wirklichkeit.

Als Lösung entwickelt die vorliegende Untersuchung einen praktikablen Ansatz für eine brückenspezifische und mehrzielige Instandhaltungsplanung. Dieser ermöglicht die Erarbeitung maßgeschneiderter Strategien zur optimalen Erfüllung bauwerksspezifischer Anforderungsprofile. Den Kern dieses Ansatzes stellt das neuentwickelte SAMS Programm dar, mit dem eine simulationsbasierte Prognose der Auswirkungen von Instandhaltungsstrategien durchgeführt werden kann. Dieses Programm beruht auf der Monte-Carlo-Simulationstechnik und berücksichtigt umfassend die wechselseitigen Abhängigkeiten zwischen Bauwerk, Schädigungsentwicklung und Instandhaltungsmaßnahmen, wie sie sich im brückenspezifischen Kontext ergeben. Als Ergebnis liefert es Häufigkeitsverteilungen für Kennzahlen, die das strategieabhängige Leistungsvermögen von Brücken bezüglich der Aspekte Zustand, Tragsicherheit, Wirtschaftlichkeit, Gesellschaft und Kultur beschreiben. Wirkungsvoll kombiniert wird das SAMS Programm mit einer eigens hierfür entwickelten Methode für eine unverzerrte mehrzielige Strategiebewertung. Ergänzend kommt ein Fragebogen zur brückenspezifischen Identifizierung von Anforderungen zum Einsatz, der für die Datenerhebung in Hamburg und Chennai erstellt wurde. Eine experimentelle Anwendung dieses Ansatzes im Rahmen zweier Fallstudien zeigt, dass bei den betrachteten Brückenbauwerken geeignete Instandhaltungsstrategien identifiziert werden können, um das brückenspezifische Anforderungsspektrum zu erfüllen. Ebenfalls zeigt eine vergleichende Betrachtung, dass der brückenspezifische Kontext entscheidenden Einfluss darauf hat, in welchem Umfang Strategien ihre spezifischen Stärken ausspielen können.

# Contents


1 Introduction ..... 1

1.1 Problem Definition ..... 1

1.2 Research Objective and Thesis Overview ..... 1

1.3 Scope and Limitations ..... 2

2 Foundations of Bridge Management ..... 3

2.1 Introduction ..... 3

2.2 Bridge Management and Bridge Management Systems (BMS) ..... 3

2.3 Bridge Performance and Performance Indicators ..... 5

2.4 Inspection, Assessment, and Evaluation of Bridges ..... 7

2.5 Preferences – Demands, Requirements, Objectives ..... 16

2.6 Bridge Maintenance ..... 29

2.7 Assessment of Maintenance Strategies ..... 38

2.8 Evaluation of Maintenance Strategies ..... 52

2.9 Conclusion ..... 61

3 Survey of Bridge Management Practice ..... 63

3.1 Introduction ..... 63

3.2 Scope and Instruments of the Bridge Survey ..... 63

3.3 Bridge Specific Demands, Requirements, and Maintenance Objectives ..... 80

3.4 Bridge Specific Maintenance Practice ..... 87

3.5 Practical Approaches to Systematic Maintenance Planning ..... 91

3.6 Current State of Bridge Management Practice in Hamburg and Chennai ..... 92

3.7 Conclusion ..... 94

4 A New Approach to Multi-Objective Maintenance Planning ..... 95

4.1 Introduction ..... 95

4.2 General Description of the Approach ..... 95

4.3 Structural Representation ..... 97

4.4 Deterioration Mechanisms ..... 114

4.5 Maintenance Activities ..... 137

4.6 SAMS-Model Adjustment to Bridge Inspection and Assessment Results ..... 154

4.7 Performance Indicators ..... 156

4.8 Maintenance Strategies ... 167
4.9 Strategy Assessment and Evaluation ... 168
4.10 SAMS Program Structure ... 171
4.11 Conclusion ... 174
5 Experimental Strategy Assessment and Evaluation ... 177
5.1 Introduction ... 177
5.2 Case Study Bridges ... 177
5.3 Identification of Bridge Specific Preference Profiles ... 181
5.4 Structural Representation ... 182
5.5 Deterioration Mechanisms and Maintenance Activities ... 188
5.6 Maintenance Strategies ... 220
5.7 Simulation-based Strategy Assessment ... 233
5.8 Multi-Objective Strategy Evaluation ... 287
5.9 Strategy Improvement ... 298
5.10 Contribution of the Approach ... 305
5.11 Conclusion ... 306
6 Multi-Objective Bridge Maintenance – A New Perspective ... 307
6.1 Introduction ... 307
6.2 Research Needs and Starting Point of the Investigation ... 307
6.3 Evidence for Bridge Specific Requirements and Maintenance Application ... 309
6.4 New Tools for the Requirements/Objectives Development Phase ... 309
6.5 Approach to Multi-Objective Maintenance Planning ... 310
6.6 Experimental Application of the Approach ... 311
6.7 Practical Implications and Outlook for Future Research ... 312
References ... 307
Symbols and Notation ... 335
Appendix ... 339
Appendix I ... 341
Appendix II ... 351
Appendix III ... 357
Curriculum Vitae ... 379

# 1 Introduction

## 1.1 Problem Definition

Bridges affect many aspects of everyday life and thus have to fulfil various requirements. In their function as traffic links, for example, they decisively contribute to the economic performance of regions and countries. Furthermore, as eminent parts of the built environment they often draw attention and—especially in urbanised areas—they diversely interact with their surroundings. In addition, they can constitute a part of the cultural heritage, especially in the case of masterpieces in shaping and design or witnesses of engineering history. In essence, bridges have to fulfil a broad range of requirements in the areas of function, economy, society and culture, as well as environment.

Bridges are long-term assets with a planned service life of around 100 years; therefore it is quite clear that they have to be continuously maintained in order to fulfil those many different requirements. Because of this, maintenance strategies should be selected with respect to maintenance objectives that account for the bridge specific context. However, current research and bridge maintenance practice still only consider the minimisation of service life costs and the fulfilment of minimum functional requirements as the primary objectives for the planning of maintenance strategies—even though the general awareness for multi-objective maintenance planning has risen over the last years. Such monothematic approaches may be helpful for bridges that are part of a redundant network. However, for large or otherwise important bridges other objectives often become decisive. In the case of a major fixed link or a bridge within a high-speed railway network, for instance, the maximisation of the traffic availability can outweigh maintenance costs. Furthermore, some structures require a maximisation of service life even at high maintenance costs—a case which typically occurs with structures of historic interest.

## 1.2 Research Objective and Thesis Overview

The present investigation aims to contribute to a better understanding of the implications, which the bridge specific context and the manifold performance aspects have on maintenance planning decisions. In addition, it proposes an innovative practical approach and related tools that can be applied to strategic maintenance planning. By this, it wants to increase the awareness for bridge specific and multi-objective maintenance planning in practical bridge management and also lay the foundation for further research and development activities in this field.

The investigation is divided into three major parts. The first part (chapter 2) refers to the actual state of the art in research and maintenance practice. It introduces the ideal bridge specific and multi-objective maintenance planning approach as postulated in the relevant literature, and—as opposed to this—shows the actual lack of such an approach both in bridge management practice and in theoretical investigations. It also contains the proposal of a simple approach for multi-objective maintenance strategy evaluation. The second part (chapter 3) introduces a questionnaire for the identification of bridge specific requirements and provides empiric evidence for the distinctness of these requirements. This is done by means of a questionnaire/interview survey of bridge maintenance practice, which involved four major bridge authorities in Hamburg (Germany) and Chennai (India). The third part (chapter 4–5) describes the development of an innovative instrument for strategic maintenance planning in the form of a program for simulation-based assessment of maintenance strategies (SAMS). In addition, it investigates the particularities of multi-objective maintenance planning in the bridge specific context. This is done with the help of a comprehensive simulation study, which uses the example of two case study bridges. In a final synopsis (chapter 6), the thesis puts into context the three major parts and on this basis describes a new perspective to strategic bridge maintenance planning.

## 1.3 Scope and Limitations

The present study focuses on strategic maintenance planning for bridges. Closely related topics such as overall bridge management or bridge inspection are only briefly looked at.

Although the general approach of the study is not limited to certain types of bridge structures, a detailed look is taken on the maintenance of prestressed concrete bridges for car traffic. The introducing investigations—the survey, the development of an evaluation approach and the questionnaire—include the whole spectrum of structural types: prestressed or reinforced concrete bridges or structures made of steel or masonry. As opposed to this, the SAMS-program in its current layout as well as the further in-depth simulation studies both refer to prestressed concrete bridges, which carry car traffic.

Because the study takes a strategic perspective on maintenance planning, the model representation and the assessment by the SAMS-program reasonably relies on simplifying and idealising assumptions. This implies first that the model representation with respect to structural analysis uses idealised cross sections and externally calculated internal forces, which are not affected by deterioration and maintenance activities. Second, the representation of the bridge equipment relies on an idealised condition index rating. Third, the deterioration and maintenance effects have been estimated or deduced from bridge inspections by applying simplifying interpretations of the qualitative inspection data. Consequently, it must be kept in mind that the numerical results of the simulation study in chapter 5 are not exact in terms of their absolute values. Instead they offer a basis for comparing alternative strategies.

# 2 Foundations of Bridge Management

## 2.1 Introduction

The present chapter provides an overview on the general bridge management process by defining and exemplifying the commonly applied terminology and concepts. Furthermore, the present chapter summarises and discusses the actual state of the art of bridge management and maintenance planning, as it is described in the relevant literature. In doing so, it highlights fields for potential improvement, and thus lays the foundation for the further investigation in the scope of the present thesis.

## 2.2 Bridge Management and Bridge Management Systems (BMS)

### 2.2.1 Bridge Management

The relevant literature defines the term *bridge management* as an object-oriented process of planning and coordination of a multitude of activities with the aim that a bridge performs during its service life in a way that is desired by the bridge authority and that is required by mandatory standards, guidelines and codes of practice. (fib 2008a: 119; fib 2002: 96; OECD 1992: 17; Ryall 2010: 3–4, Haardt 2002: 16). The Draft Model Code 2010 uses the alternative term *conservation management* (fib 2010b: 264–267). However, the term *conservation* will not be used in this thesis in order to avoid confusion with the connotative *preservation* of historic assets (e.g. Hornby and Turnbull 2010: 320; Tilly 2002). Moreover, bridge management—and not conservation management—is the widely used expression as can be seen from the multitude of titles of conferences, books and papers regarding this topic.

According to the above mentioned literature, the bridge management process comprises the consecutive activities *bridge inspection*, *bridge assessment*, *bridge evaluation*, *strategic maintenance planning*, *operational maintenance planning* and finally tendering and supervising the *execution of works*. The overall process is conducted under consideration of *performance requirements*, which the bridge must fulfil, and with a view to regulations laid down in *standards* and *codes of practice* or *guidelines*, as well as to *maintenance objectives*, which are formulated by the bridge managers.

The bridge inspection activities collect raw data, such as defects or ongoing deterioration processes. The bridge assessment activities encompass analysing these raw data and interpreting them in order to describe the current condition as well as to predict the future performance development. The subsequent bridge evaluation verifies that the assessment results are in accordance with defined performance requirements. Based on these evaluation results, maintenance activities are planned on a strategic and operational level. Finally, the management team organises the execution of the planned maintenance activities.

The flowchart in Fig. 2.1 illustrates the bridge management process as it is defined in this thesis on the basis of the *fib* Draft Model Code 2010 (fib 2010a: 27–28; fib 2010b: 264–267), the reports of the *fib* Task Group 5.3 (fib 2008a: 22 and 68), and the FIP Commission 10 (fib 2002: 9).

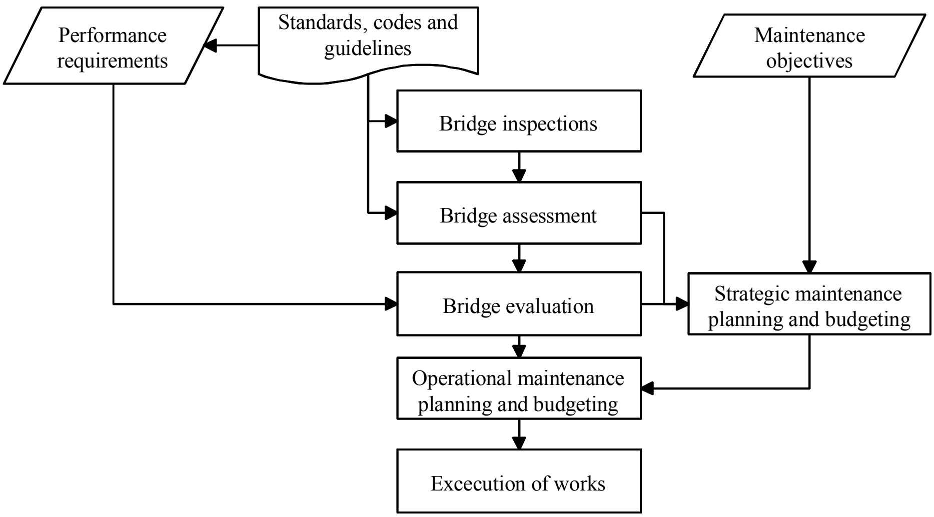



**Fig. 2.1:** Bridge management: Related activities and process flow (based on fib 2008a: 22, Figure 25 and fib 2010b: 267, Figure 9.3-2)

### 2.2.2 Bridge Management Systems (BMS)

Modern *bridge management systems* (*BMS*) are software based systems that are used for inspection, assessment, evaluation, strategy selection and maintenance planning for bridges. These computerised systems usually comprise an *inventory data base*, which contains technical data of the bridges, and an *inspection data base* with the findings of the bridge inspections (Mehlhorn 2007: 973–974; OECD 1992: 95–97). These two modules support the first three bridge management activities (inspection, assessment and evaluation) by providing data for inspection planning as well as storing and processing the inspection data in order to assess and evaluate the bridges. An example for this is the German *SIB-Bauwerke,* which provides these two features (Holst 2005: 16–17). Further developed Bridge Management Systems comprise additional data bases and modules for supporting strategy selection and maintenance planning (Mehlhorn 2007: 973–974). The German *BMS*, for example, provides modules for estimating the future development of the bridge condition and deterioration prediction as well as for forecasting of maintenance costs and budgeting (Holst 2005: 41). Moreover, this BMS also provides modules for the assessment and optimisation of maintenance strategies.

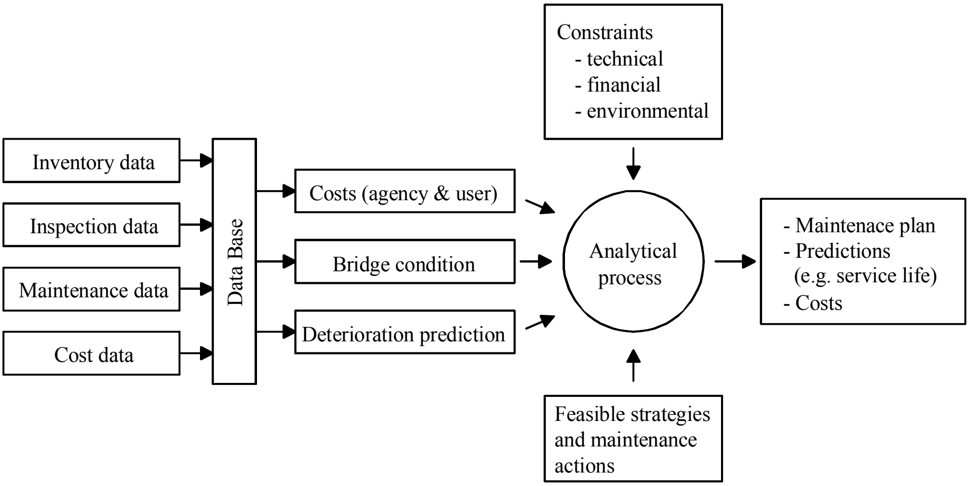



**Fig. 2.2:** BMS prototype (based on OECD 1992: 97, minor alterations).

The above Fig. 2.2 illustrates the long-known and still valid prototype structure for Bridge Management Systems, which is described by the report on the research and experience of OECD member countries in the field of bridge management (OECD 1992: 97). Today many different computerised systems for the support of bridge management activities are applied or under development, for example *DANBRO* in Denmark, *SIB-Bauwerke* and *BMS* in Germany, *KUBA* in Switzerland, *SMIS* in the United Kingdom or *Pontis* and *BRIDGIT* in the United States (Hearn et al. 2005; Haardt 2002, Tonias and Zhao 2007: 471).

## 2.3 Bridge Performance and Performance Indicators

The terms *performance* and *performance indicators* are central and recurring terms in bridge management. The fib Draft Model Code 2010 uses the term performance in the meaning of '*the behaviour of a structure or a structural element as a consequence of actions to which it is subjected or which it generates*' (fib 2010a: 16). In the case of bridges, this kind of behaviour has many different facets because bridges interact with their surroundings in various ways—passively and actively. Passively they respond to loads and deterioration attack, for example in terms of deformation or damage accumulation. Actively they affect their societal, cultural end ecologic environment—for example as means of transport, as costly public assets, as widely visible structures, and also as producers of greenhouse gas emissions and waste during construction and demolition. Altogether, these interactions can be allocated to four distinct fields: *function*, *economy*, *society and culture*, and *environment*. Consequently, the Draft Model Code 2010 uses the term *performance aspect* in order to differentiate between these different ways of active and passive behaviour. On the basis of these considerations, the following definitions will be used in the scope of the present thesis:

**Definition 1**: The term **bridge performance** describes the passive and active behaviour of a bridge. A bridge responds passively to loads and deterioration mechanisms, whereas it interacts in an active way with its surroundings by providing service to the public, requiring public funding, being widely visible, and having an ecological impact.

**Definition 2**: The term **performance aspect** is used to refer to the different ways in which a bridge behaves. Example performance aspects are: safety, maintenance costs, aesthetic appearance, traffic capacity, climatic impact and waste production. In general, the performance aspects refer to four separate fields: function, economy, society and culture, and environment.

However, bridge performance is a rather vague term. Therefore performance indicators must be introduced in order to quantitatively describe the different functional, economic, societal/cultural and economic performance aspects. Such indicators are measurable and/or computable parameters, which quantitatively describe the condition of the structure and the impact on the surroundings (cp. fib 2010a: 16). Measurable parameters are, for instance, the maximum deflection of a bridge or the traffic flow, while the cross sectional resistance is an exclusively computable parameter (except for the case of destructive testing).

The *functional performance indicators* describing structural safety, serviceability and durability are well known and explicit definitions can be found in the relevant standards and codes of practice. Example indicators are the reliability index β, stresses in concrete and steel, crack widths, deflections and vibrations as well as carbonation depth and or chloride content.

Also many *economic performance indicators* are widely used in engineering practice and research. These refer usually to the owner costs—in the form of expenses for design, construction, maintenance, decommissioning—and to the costs for the users of the bridge or other monetarised adverse effects on the environment. Example indicators are the *net present value* of the costs and the *annuity* (see also equation 2.9 and 2.10).

The *environmental performance indictors* usually specify the environmental impact with respect to greenhouse gas emission, resource consumption, waste generation and other environmental impacts such as eutrophication. These aspects are represented by performance indicators, such as *$CO_2$ equivalent emissions* (kg), *primary energy consumption* (J), *landfill* ($m^3$) and *$PO_4$ equivalent* (kg).

However, there is a lack of *societal and cultural performance indicators*, which quantitatively describe the performance of bridges with respect to their active interaction with societal and cultural issues. On the one hand, the actual relevant literature in general recognises the societal and cultural performance aspects as being important, such as the fib Draft Model Code 2010, which explicitly refers to general societal and aesthetic performance aspects (fib 2010a: 43–47). However, on the other hand no detailed performance indicators are found. Useful in this field would be indicators that measure the direct impact of the structure on the users' and residents' quality of living, e.g. aesthetic appearance, or their general impact as landmarks, monumental sites and heritage structures.

In summary, the following definition—based on the definition by the fib Model Code 2010—will be used in the scope of this thesis:

> **Definition 3**: The term **performance indicator** describes a measurable or computable parameter, which quantitatively describes a performance aspect.

## 2.4 Inspection, Assessment, and Evaluation of Bridges

### 2.4.1 Overview

Bridge inspection is generally conducted on three levels of sophistication. *Superficial inspections* are conducted in close time intervals in order to spot defects that require urgent interventions, while *principal inspections*, in the form of *general* and *major inspections*, are conducted in medium term intervals in order to assess the actual condition of the structure. The *special inspections* comprise various kinds of in-depth investigations in order to gain detailed insight in the structural condition and deterioration development.

The information gathered during the inspections is interpreted during the assessment phase in the form of an overall condition assessment, which regularly also includes the calculation of indices for condition rating, and sometimes even in the form of detailed theoretical recalculations or experimental measurements of material properties.

The evaluation phase compares the assessment results with the defined performance requirements which the bridge must fulfil. Based on these evaluation and assessment results the whole range of maintenance planning activities is conducted: decisions for urgent interventions as well as the operational and strategic planning.

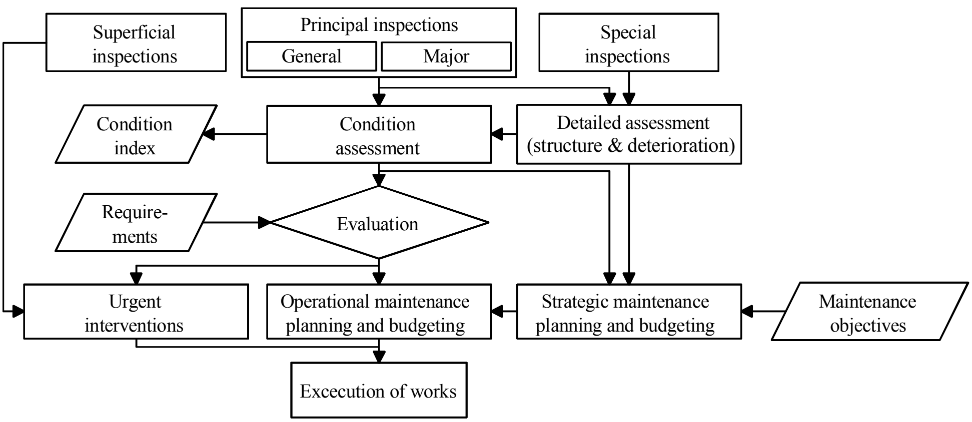



**Fig. 2.3:** Inspection, assessment, and evaluation (*Note*: This figure is based on fib 2008a: Fig. 2–1 and fib 2002: Fig. 25):

### 2.4.2 Bridge Inspections

It is common practice that bridge authorities inspect the bridges under their responsibility on a regular basis for the following purposes (TRB 2007: 48; Ponnuswamy 2009: 544–547; fib 2008a: 63–66; fib 2010b: 272–276; OSA 2004; Ryall 2010: 45):

- immediate detection of damages that necessitate urgent interventions
- identification of the condition in order to find out medium-term maintenance needs
- recognition of long-term major rehabilitation or upgrade needs and identification of long-term development trends in order to support strategic maintenance planning
- in-depth assessment for further investigation of major damage or after extreme loads, such as after an earthquake

Bridge authorities apply different types of inspections that account for these different purposes. These inspection types differ in terms of the time intervals, the intensities, the deployed personnel, and the provided results. A characteristic differentiation into four different types is common practice (Mehlhorn 2007: 948–949; COST 2002a: 52–53; OECD 1992: 23–24):

- superficial inspections
- general inspections
- major inspections
- special inspections

*Superficial inspections* are conducted with the above mentioned purpose of immediate detection of safety-relevant damages, which necessitate urgent interventions in order to ensure the continuous safety of the bridge, both in structural and traffic related terms (DIN 1076: 5, TRB 2007: 48). They are usually carried out by regular road maintenance personnel in short-term intervals. These time intervals range from three month in Germany and the UK to yearly inspection intervals in Sweden (TRB 2007: 49; DIN 1076: 5; LISt 1997: 2; Ponnuswamy 2009: 546). The superficial inspections are carried out in the form of quick visual checks for new defects or a significantly altered bridge condition. For example, this can be extraordinary deformations, impact damage, major defects of the road surface and damaged expansion joints (DIN 1076: 5, Mehlhorn 2007: 949; TRB 2007).

*General inspections* are conducted in order to detect defects and deterioration processes that require medium-term interventions, for example maintenance of the asphalt pavement or patch repair of the concrete surface and of the steel-painting. They generally comprise an intensive visual inspection of the load-bearing structure as well as of the bridge equipment, such as bearings and expansion joints (DIN 1076: 5; TRB 2007: 48; OECD 1992: 24). However, heavy inspection equipment is not applied as opposed to the below described thorough major inspections; consequently only directly accessible elements can be investigated in detail (DIN 1076: 5; TRB 2007: 48; OECD 1992: 24). The general inspections are carried out by specially trained personnel—usually with a background in bridge engineering or by certified technicians—in intervals ranging from one year in the United States and India to three years in France and Germany (DIN 1076: 5; TRB 2007: 48–49; OECD 1992: 24; COST 2002a: 52; IRC 1996: 15). The results are recorded in an inspection log including descriptions of the found defects, mid-term repair needs, recommendations for further detailed inspections and an updated bridge condition rating (COST 2002a: 52; TRB 2007: 49).

*Major inspections* provide a comprehensive overview on the condition of the bridge in order to recognise long-term rehabilitation or upgrade needs and to support strategic maintenance planning (TRB 2007: 48; fib 2010b: 274; Ryall 2010: 45). The major inspections are characterised by intensive examinations of all parts of the structure at arm's length with the help of heavy inspection equipment that provides access to usually inaccessible parts, as for example truck-mounted platforms (DIN 1076: 3; COST 2002a: 52; fib 2010b: 274). These inspections are carried out by experienced and specially trained bridge engineers in intervals ranging from five years in Norway and South Africa to ten years in the United States (TRB 2007: 49; COST 2002a: 52). In their results, the major inspections provide detailed descriptions of the defects and a resulting overall condition rating of the bridge as well as detailed recommendations for mid-term maintenance needs and long-term rehabilitation or upgrade needs; they also include estimates of quantities and costs (TRB 2007: 49; COST 2002a: 52; DIN 1076: 3; RI-EBW-PRÜF 2007, 2013). Based on these results, long-term development trends regarding the bridge condition—a very valuable information for the strategic maintenance planning—can be identified by comparison with earlier inspection results (fib 2010b: 274; Ryall 2010: 45).

*Special inspections* are conducted when there is need for more detailed information than is provided by the abovementioned regular inspections. This, for example, is the case if imminent threats to structural safety or serviceability are assumed, as they can result from major defects or from accidental overloading caused by extreme events. Special inspections are also conducted in order to gain detailed information on deterioration mechanisms, for example corrosion rates, or in order to prepare major rehabilitation works. Various specialised inspection equipment and procedures are usually applied for recovering those detailed information. This can be for instance: core drilling, sophisticated non-destructive testing or even test loading of the structure in connection with deformation measurements. Due to the specialised procedures these inspections are often conducted involving external experts from engineering consultants or research institutions. (OSA 2004; fib 2008a: 65–66; fib 2010b: 274–275; COST 2002a: 53; TRB 2007: 48–49)

### 2.4.3 Bridge Assessment – General Definition

The inspections provide information in the form of raw data, such as defect listings or more specific data, such as chloride concentration or deformations under proof loadings. These raw data do neither provide a picture on the overall condition of the structure and its actual performance nor do they reveal long-term development trends, such as the expected deterioration progress. However, such information is vital for an effective bridge management. Therefore, the findings of the inspections must be interpreted, processed and represented in suitable form.

The term *assessment* is used in the relevant literature to describe processes in connection with analysing the inspection results. The Draft Model Code 2010, for instance, uses the term *condition assessment* to describe a '*process of reviewing information gathered about the current condition of a structure or its components, its service environment and general circumstances*' (fib 2010a: 6). This assessment is conducted in order to '*obtain an indication of current performance and to make a prognosis of future performance, including identification of deterioration mechanisms and prediction of damage*' (fib 2010b: 264). Corresponding definitions are used by the fib bulletin *Concrete structure management: Guide to ownership and good practice* (fib 2008a: 119) and by the FIP Commission 10 (fib 2002: 96).

The above *fib*-publications use the term assessment as opposed to *evaluation*—the latter term describes the process of comparing the interpreted inspection results to defined performance requirements (e.g. fib 2010a: 7; fib 2008a: 92). Such an explicit separation of terms is not followed, for example, by the Swiss standard *SIA 260,* which uses the term *condition evaluation* as a collective term encompassing both assessment and evaluation. Also, the UK Department of Transport uses the term assessment exclusively to describe the evaluation of structural safety by referring to a '*process of confirming the adequacy of a structure to support specified loads'* (DfT 2005: 31).

The term *condition* describes the overall physical state of the bridge, as for example structural integrity and defects in bridge equipment, overall appearance and ongoing deterioration mechanisms—but also the theoretically derived structural safety. The term *condition assessment* is also used by the above *fib*-publication. It refers to all relevant performance aspects and encompasses the description of structural performance—safety, serviceability, durability—and the description of the overall deterioration level as well as detailed investigations and descriptions of deterioration mechanisms (fib 2010b: 277–278).

However, other authors in this field do not follow this all-embracing usage of the term condition and condition assessment. The report of the *COST 345* cross-country survey on procedures for the assessment of highway bridges, for example, clearly differentiates between condition assessment, which is limited to the description of the physical properties, on the one hand and *structural assessment*, which refers to safety and serviceability, on the other hand (COST 2002a, 2002b). An analogous differentiation is made by Ryall (2010).

This differentiation is also common bridge management practice; it usually relies on visual *condition rating* and applies calculations of structural safety only in cases of major concern. The currently applied German bridge assessment software tool *SIB-Bauwerke*, for instance, applies a condition rating that is calculated based on all detected defects, which are nominally rated in terms of structural safety, traffic safety and durability on a five-point scale (RI-EBW-PRÜF 2007, 2013; SIB 2011). However, the ratings regarding these three aspects are usually taken directly from a list of so called *Schadensbeispiele* (example defects) of the RI-EBW-PRÜF, which offers numerical ratings for typical defects (RI-EBW-PRÜF 2007, 2013; SIB 2011). Otherwise, if a suitable example defect is not found in the list, the ratings are usually based on educated guesses of the bridge manager. Consequently, the ratings—especially regarding the effects on structural safety—are usually of rather intuitive kind; this is also found by Frangopol and Liu (2007: 31). In order to account for this, the German guidelines for the management of federal highways require a detailed structural assessment in cases of major concern (OSA 2004). The regular bridge management practice therefore effectively differentiates between simple condition assessment and more sophisticated structural assessment.

In conclusion to the above debate, the term *bridge assessment* is defined in the scope of the present thesis according to the definition by the Draft Model Code 2010 as a process of inspection data analysis. Also, the assessment is clearly distinguished from evaluation, which is used to describe a comparison of the assessment results with defined requirements. In contrast to the *fib*-publications, a subdivision into three separately labelled assessment activities is also considered in the scope of the present thesis:

- *condition assessment* being a basic and rather qualitative interpretation of the inspection results by a condition rating system which applies a *condition index*
- *structural assessment* being a detailed numerical assessment of structural safety based on the findings from the inspections
- *deterioration assessment* being a detailed analysis of selected defects and deterioration processes in terms of causes, extent, implications on performance, and future development

Thus the following definition will be used in the scope of this thesis:

> **Definition 4**: The term **bridge assessment** describes the overall process of interpreting, processing and representing the information gathered during the bridge inspections in order to obtain an indication of current performance as well as to predict the future developments. The overall bridge assessment process encompasses three distinct assessment activities: (1) condition assessment (condition rating), (2) structural assessment, and (3) deterioration assessment. Bridge assessment is clearly distinguished from bridge evaluation as it exclusively indicates and predicts the performance but does not compare it to requirements and objectives.

### 2.4.4 Condition Assessment (Condition Rating and Condition Index)

Bridge authorities all over the world apply *condition rating systems*, which support their maintenance planning by providing indication of the bridge performance (BRIME 1999: 53–55). The condition rating system usually applies a numerical rating indicator, for example in the range from 0 to 4 for individual defects and from 1 to 4 for the overall bridge condition in Germany , which is calculated based on ratings of single defects that are found during the bridge inspections (Hearn et al. 2005: xvi, RI-EBW-PRÜF 2007, 2013). Examples for international rating systems are depicted in Table 2.1.

**Table 2.1:** Individual defect rating (international practice)

| Country/City | Defect rating | Rating aspects |
|---|---|---|
| Finland | 0 (best) – 4 (worst) | Damage effect on load capacity , urgency of repair, severity and overall condition of the damaged element (apart from the rated defect) |
| Germany | 0 (best) – 4 (worst) | Damage effects on load capacity (S), traffic safety (V), and on durability (D) with explicit rules for superposition of (S, V, D) |
| New York City | 7 (best) – 1 (worst) | Severity and effect on element functionality |

*Note*: This table is based on Table 2 in Hearn et al. (2005: xvi) with additional information from: Hearn et al. 2005: 13; Haardt 1999: 39; Ryall 2010: 88–89; Dubin and Yanev 2001: 6

In the following, the indicator that describes the condition—both regarding the condition of the overall bridge and bridge elements as well as single defects—is labelled *condition index*. Such a condition index provides an easy to handle indicator that can be used for the following purposes (Haardt 1999: 11; BRIME 1999: 54; COST 2002a: 89):

- indication of weak points and special threats
- indication of critical developments (for single bridges and for the overall bridge stock)
- decision support for bridge specific maintenance (prioritisation, scheduling)
- decision support for resource allocation within the bridge stock (prioritisation)
- rough estimate for deterioration development (comparison of sequential periods)

In bridge assessment practice the actual calculation of the overall condition index is achieved by three alternative approaches, which can be labelled as (1) *worst element classification*, (2) *cumulative condition rating*, and (3) *weighted condition rating*. This categorisation into three approaches follows from several reports on cross-country surveys on bridge management practice (COST 2002a: 90; BRIME 1999: 55; Hearn et al. 2005; Ryall 2010: 86–91).

The *worst element classification* calculates the bridge condition index based on the worst single rating of all bridge components. An example for this is the German calculation approach, which is implemented in the software for bridge assessment *SIB-Bauwerke*. This approach calculates the resulting bridge condition index on the basis of the worst single rating. It considers three corrective terms that account for the extent of damage with respect to the single defect types, to the importance of the damaged elements, and to the overall damage extent (Haardt 1999: 14–15, 36–39).

The *cumulative condition rating* calculates the bridge condition index from the sum of single defect ratings. An example for this is the Finnish rating system, which calculates the *KTI Repair Index* as a weighted sum of the ratings of all structural bridge parts. This algorithm takes into account the structural importance of bridge elements, the urgency of repair and the severity of the damage (Hearn et al. 2005: 13).

The *weighted average condition rating* calculates the bridge condition index from all detected defects and their individual rating. The weightings usually represent the relative importance of the affected bridge elements. An example for this is the New York City rating system. This system applies thirteen individual weighting-factors in order to account for the different relevance of elements such as '*primary members of the deck'*, which are weighted highest with 10, and elements such as kerbs and sidewalks with the lowest weightings 1 and 2 respectively (Ryall 2010: 88–89; Dubin and Yanev 2001: 6; Yanev and Testa 2000: C5/5–C5/7). A similar method for a weighted average condition rating is proposed in *Heft 572* of the DAfStb for providing a condition indicator for maintenance planning (DAfStb 2007: 93).

In conclusion, the following definition for the term *condition rating* is used in the scope of this thesis.

> **Definition 5**: The term **condition rating** refers to the process of describing the overall physical state of the bridge—structural integrity and defects in bridge equipment, overall appearance and ongoing deterioration mechanisms—with the help of an indicator (**condition index**) that regards the condition of the overall bridge and of bridge elements as well as single defects.

### 2.4.5 Structural Assessment

After taking a bridge into service, detailed structural analyses are not conducted on a routine basis, in contrast to the regular condition rating. Naturally, only special situations justify—or necessitate—the considerable amount of work that is connected to a detailed structural recalculation of an existing bridge. Typical causes for conducting a structural analysis of an existing bridge are the following three issues (fib 2002: 15, 49; COST 2002b: 3; Albertin-Hummel and Brandt 2007):

- detection of defects or deterioration processes that might pose an imminent threat to structural safety (e.g. loss in cross-sectional area or decrease in strength properties)
- increased loads (e.g. traffic loads or seismic loading) or change in use
- new findings about potential threats (e.g. stress corrosion of prestressing tendons)

The detailed re-calculation of a bridge serves the following purposes (fib 2002: 49; COST 2002b: 3):

- calculation of the actual load carrying capacity and the internal forces
- diagnosis of structural defects (e.g. causes of cracks)
- design and preparation of rehabilitation and strengthening measures

The input data, such as loads and material properties, as well as the applied methods of structural analysis are usually adjusted to the special circumstances and purpose of the structural assessment. Common practice is to apply increasing levels of sophistication: from simple re-calculation based on current standards, to sophisticated (e.g. non-linear) analysis and usage of in-situ material data and load testing, up to a direct reliability analysis (BMVBS 2011: 10–12; Ril 805: Modul 805.0001, pp. 3–4; SIA 269; COST 2002b: 4). The following list shows, for example, the increasing levels of sophistication as they are applied in the newly introduced German guideline for the structural assessment of existing road bridges (BMVBS 2011: 10–12):

- level 1: re-calculation of loads and resistances according to current standards
- level 2: application of complementary rules to the current standards (e.g. regarding the minimum angle of the inclination of the compressive stresses in shear)
- level 3: application of load tests in order to gain insight in the real structural behaviour and to validate the models for structural analysis
- level 4: application of scientific methods for assessment, e.g. sophisticated non-linear modelling and direct reliability analysis

In conclusion, the following definition for the term *structural assessment* is used in the scope of this thesis. This definition is based on two different definitions by the *fib*—the definition for *structural assessment* (fib 2002: 15) and for *recalculation* (fib 2010a: 18).

> **Definition 6**: **Structural assessment** is defined as the analysis of an existing structure (or parts of it) in order to describe its structural performance in terms of the load bearing behaviour and resistances. This analysis may comprise similar steps and procedures to the design of new structures. However, it differs from this by seeking to take into account the actual form and condition of the structure, including ongoing or potential deterioration processes. Structural assessment usually becomes necessary if the inspection results or new scientific knowledge—defects, deterioration mechanisms or other potential threats—indicate that the structure may no longer be capable of fully undertaking its required function with the necessary level of safety. Structural assessment may also be indicated when design loadings are revised (e.g. earthquake, temperature) or when loads increase

(e.g. permitted maximum size of truck or axle weight). In its results, it provides data on the actual load carrying capacity and internal forces. It also may provide explanations for structural defects (e.g. causes of cracks) or it may be conducted in connection with the design and preparation of rehabilitation and strengthening measures.

### 2.4.6 Deterioration Assessment

Detailed analyses of deterioration data can become necessary because of many reasons. One reason can be that the findings from the visual routine inspections indicate the necessity for more detailed investigation of causes, extent and implications of defects that are potentially critical or otherwise important. Also, a structural assessment may be intended, which needs more detailed data on the actual structural condition than is provided by the listings of defects and the derived condition ratings. And finally, a prognosis of the future development may be desired, for example in order to support long term maintenance planning tasks. Typical examples for deterioration assessment activities are investigations on the degree and rate of corrosion of reinforcement as well as assessing the remaining prestressing forces in tendons (fib 2008a: 67; fib 2002: 18). Detailed deterioration assessment is indicated in the following cases (e.g. OSA 2004: 11–13):

- *unclear causes* of defects (e.g. cracks and deformations)
- *suspected* defects (e.g. suspected corrosion of transverse tendons in the deck slab)
- *unclear effects* of defects (e.g. leakages with unknown effects like tendon corrosion)
- *unclear extent* of defects (e.g. carbonation depth, chloride concentration)
- *unexpected development* (e.g. quicker than expected development of cracks and spalling)
- *prognosis of future development* (e.g. expected development of chloride concentration)

In conclusion, the following definition for the term *deterioration assessment* is used in the scope of this thesis.

> **Definition 7**: The term **deterioration assessment** describes activities that encompass detailed analyses of defects and deterioration processes in order to gain information on causes and the extent as well as on the implications on the actual bridge performance and also to predict future developments.

### 2.4.7 Bridge Evaluation

The bridge evaluation process states how well the actual performance, as it is found by bridge the assessment activities, complies with the expectations and requirements of the affected parties.

On the one hand, bridge evaluation is achieved by comparing the actual performance with explicitly defined (minimum) requirements. These requirements are defined in the form of limits for the assessed performance aspects (fib 2010a: 7; fib 2008a: 122) and are usually provided by national and international standards, such as the German DIN or the ISO, or other official regulations and codes of practice, such as the German RI-EBW-PRÜF. Typical examples are maximum admissible crack width, a maximum admissible probability of failure or regulations referring to waste management and noise emissions

On the other hand, bridge evaluation is also achieved with the help of gradual interpretative valuation of the assessment results. This is typically the case with general bridge condition rating, where the condition indicator is interpreted with respect to a preference scale, ranging for example from ‘excellent’ and ‘good’ to ‘critical’ (e.g. RI-EBW-PRÜF 2007, 2013). In contrast to the above described minimum requirements, this kind of evaluation refers to a preferred tendency rather than to an explicitly defined border between the desired and adverse behaviour.

The following definition for the term *bridge evaluation* is used in the scope of this thesis.

> **Definition 8**: The term **bridge evaluation** describes the valuation of the assessment results in two ways. The first is the comparison with minimum requirements with respect to explicitly defined limits. The second qualitative way refers to a preferred tendency of the measured performance rather than to an explicitly defined border between the desired and adverse behaviour.

## 2.5 Preferences – Demands, Requirements, Objectives

### 2.5.1 Overview

Bridges are built in order to fulfil the needs of various parties, but at the same time they also affect many interests. They usually serve as important traffic links and they also can constitute expensive assets. Furthermore, they often draw a lot attention due to their exposed positions when crossing obstacles and they diversely interact with their surroundings. Consequently, ‘*it is self-evident that the success of* [a bridge] *depends on how well it meets* (...) *the owner's needs and interests or those of the users*’ (fib 2008a: 3). Obviously, this must be also the case for affected groups such as local residents or the society as a whole. The present section discusses the general implications of these interactions on bridge design and maintenance.

### 2.5.2 Stakeholders and their Demands

The term *stakeholders* describes the parties that are either directly involved in the design, construction and operation or that are otherwise affected by the bridge. These stakeholders have explicit or implicit demands with respect to the role that the bridge should play, its everyday behaviour, and its service-life performance (fib 2010a: 28; Wadenpohl 2011: 33). Wadenpohl (2011: 33) describes stakeholders of infrastructure projects as individuals or organisations that have one or more of the following three characteristics:

- They are actively involved in the project.
- Their interests are affected directly or indirectly by the project.
- Their actions can influence the objectives, the processes or the results of the project.

Similar, if less detailed, definitions are provided for example by the Draft Model Code 2010 (fib 2010a: 22) and the UK Department for Transportation (DfT 2005: 31). Wadenpohl (2011) identifies in his investigation of 15 case studies the influence of stakeholders on large infrastructure projects. Also the Draft Model Code 2010 and the UK Department for Transportation provide example listings of stakeholders (fib 2010a: 22; DfT 2005: 31, 265). Based on these sources, the following twelve groups of stakeholders are found to be most important in the case of infrastructure projects:

- owner (e.g. state or municipality)
- operator (e.g. bridge authority or concessionaire)
- designers (e.g. engineers, architects, consultants)
- execution team (e.g. contractors, suppliers)
- administration (e.g. regulatory or approving authorities)
- users (e.g. commuters, local traffic participants, commerce)
- local residents
- businesses
- socio-political interest groups (e.g. political parties, citizens' initiatives)
- media
- government
- society (including society as a representative of the natural environment)

In the scope of the present thesis, however, the label *stakeholder* will exclusively be used to describe the four parties whose interests are affected by the bridge but who are neither directly involved in the project nor can directly influence its objectives, the processes or the results. These are the users, the local residents, businesses, and the society. This restriction is decided for because bridges are usually part of the public infrastructure and therefore it is their *raison d'être* to fulfil the needs of these four parties. Consequently, the crucial topics of the demands emerge at these affected stakeholders.

For simplifying purposes it is supposed that the remaining groups—the owner, the operator, and the professional team—are mainly concerned with condensing as well as communicating the demands of the affected stakeholders and eventually turning them into a usable structure. Insofar, these remaining groups act—from a simplifying point of view—as the agents of the public will. The *owner* and the *operator* occupy a special position as they are decisively involved in interpreting the stakeholders' demand, translating them into action plans for design and maintenance and finally initiating and supervising their realisation. This is especially the case with public bridge authorities, which are highly competent in design and maintenance planning and usually take over decisive functions of the owner, the operator and the designer. The *professional team* encompasses the groups that act as consulting parties and executing agents in order to support the owner and operators in realising the stakeholders' demands: designers, contractors and suppliers (fib 2008a: 9, 22; fib 2010a: 17, 55).

Further remaining groups—administration, government, interest groups, and media—are seen in the scope of this thesis as being working on a more strategic and aggregate level and rather being concerned with condensing as well as communicating the demands of the affected stakeholders to the owner and the supporting professional team. Therefore these groups—from a simplifying point of view—do not contribute to the processes of translating bridge specific demands into action plans and turning them into reality.

This constellation of stakeholders, owner & operator and the supporting team together with their interdependencies is depicted in the following Fig. 2.4.

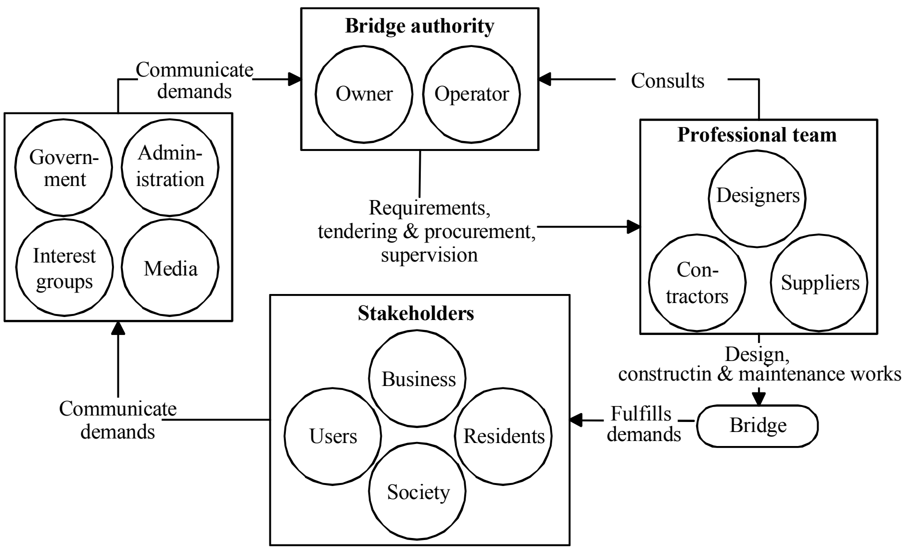



**Fig. 2.4:** Stakeholders, owner & operator (bridge authority), professional team

The interests of the stakeholders are affected directly or indirectly with respect to various aspects. In the scope of this thesis the typical aspects and associated demands are defined based on the publications of Wadenpohl and the *fib Task Group 5.3* (fib 2008a: 11 and 21; Wadenpohl 2011: 165). These authors consistently identify four categories under which all stakeholders' demands can be subsumed and which are also common terms in the sustainability debate (e.g. DAfStb 2007: 15–18):

- function
- economy
- society & culture
- environment

The *functional* aspects concern structural safety, serviceability, and durability. The society as a whole demands that structures are reasonable safe in order to guarantee for public safety as well as for the physical integrity of human beings (e.g. MBO 2008: § 3 (1); GG 2010: Art. 2 (2)). Also the society and, in particular, the users demand that the infrastructure is capable to fulfil its purpose in technical terms by providing a means of mobility. And finally, the society demands that its assets shall be constructed and maintained in such a way that its value is conserved for an appropriate period of time. A general overview is depicted in the following Fig. 2.5.

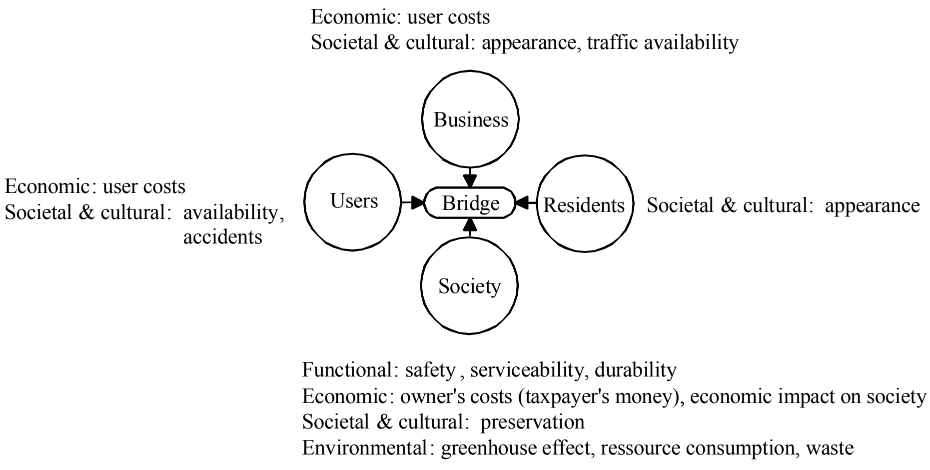



**Fig. 2.5:** Stakeholders and their demands

*Economic* issues generally concern three aspects. The first is the demand for a responsible allocation of public money for construction and maintenance of infrastructure assets. The second aspect concerns the costs to the users in connection with direct usage of the bridge, such as toll costs, which should be in balance with the benefit of the bridge. The third aspect concerns adverse economic effects on the users, the local residents, and the society as a whole including effects on the environment. An example where economic aspects—together with environmental and cultural issues—are a major public concern is the *Hochmoselübergang* in Germany. A petition of a citizens' initiative demands a halt to construction works because of the high costs, which are not seen to justify slightly lower travel times, and presumed future financial deficits of the local tourism and wine industry (Bundestag 2011). However, the petition was not successful and currently the Hochmoselübergang is being built.

The *societal/cultural* aspects concern issues of the human way of life: personal well-being, pleasant living environment, cultural identity, pursuit of prestige, and historical consciousness. In connection with the first aspect—personal well being—demands like a high availability of the traffic infrastructure or a low number of accidents emerge. A pleasant living environment demands an adequate aesthetic appearance of bridges that are located within populated areas. Finally, the demand for preservation of heritage structures emerges with the human consciousness of cultural identity and history.

Typical *environmental* demands refer to a responsible use of natural resources and to low pollution and contamination of air, water and soil as well as to waste reduction. A further major concern is the reduction of global greenhouse gas emission. Because these environmental impacts almost inevitably result from construction works and during service—they cannot be avoided at the current state of the art—the stakeholders demand usually that the negative impact shall be well balanced with the utility of the structure.

In conclusion, the following definitions for the terms *stakeholders* and *stakeholders' demands* are used in the scope of this thesis.

> **Definition 9**: The term **stakeholder** describes a member of one of the four parties whose interests are affected directly or indirectly by the in-service performance of bridges: users, local residents, business, and society.
>
> **Definition 10**: The term **stakeholders' demands** describes the demands of the four stakeholder parties (users, local residents, business, and society) which emerge from interference of the bridge with their interest or from the potential utility they gain from the structure. These demands concern functional, economic, societal & cultural, and environmental aspects.

### 2.5.3 Owner's Performance Requirements

The owner's *performance requirements* constitute the link between the demands of the stakeholders on one side and the bridge design and maintenance planning objectives on the other side (fib 2010a: 28). The owner defines these requirements by translating the various demands into specifications regarding characteristics of the bridge and its in-service behaviour (fib 2008a: 21–28). Consequently, they can be categorised into the same groups as the stakeholders' demands (fib 2008a: 27): function, economy, society & culture, and environment.

The fib Draft Model Code 2010 introduces the term *performance requirements* in order to provide explicit guidance for assessing and evaluating the behaviour of a bridge. These performance requirements are specified in terms of indicators—parameters that quantitatively describe a performance aspect—and associated criteria, which in terms of quantitative limits define the border between the desired and adverse behaviour (fib 2010a: 16). According to the definition by the fib Draft Model Code 2010, the performance requirements always constitute *minimum requirements* because the criteria mark the limits of an acceptable domain. These are for example requirements regarding deformations under service loads or the level of structural safety. In the case of these example aspects the definition of minimum requirements is perfectly suitable for expressing the stakeholders' demands—there is no obvious gain in achieving a higher level of safety or less deflection than sufficient for the intended use.

However, many performance aspects should be evaluated with respect to a preferred tendency rather than with respect to simple acceptable/non-acceptable criteria. For example, economic and societal aspects are often characterised by a direct gain for the stakeholders and owners if the performance increases, which under the name *positive marginal utility* is a well known effect in economic theory. The most obvious examples are found in lower user costs, higher traffic capacity and better aesthetic appearance of bridges. Also, a lower environmental impact is, at the end, for the benefit of all parties—even if this is sometimes less directly and immediately felt, for example in the case of reduced $CO_2$ emissions.

Typical requirements for all four performance categories, as they are found in the relevant literature, are discussed in the following:

The *functional* aspects concern the structural safety, the serviceability, and the durability. Also performance requirements with respect to the overall bridge condition rating belong to the functional domain. The requirements with respect to structural safety are usually explicitly defined by codes and standards. The required structural safety is typically expressed by the relation of the design resistance ($R_d$) to the corresponding sectional force ($S_d$). With respect to the aspects serviceability und durability, the relevant codes and standards usually only suggest criteria in terms of quantitative limits—such as maximum allowable stresses in concrete and steel ($\sigma_d$), crack widths ($w_d$), and deflections ($f_d$)—and thus leave room for choice in order to meet special demands. The requirements with respect to the bridge condition rating are even less strictly defined in terms of generally preferably high condition rating with optionally defined targeted rating levels that should, but not must, be fulfilled.

The *economic* aspects are described by the relevant literature in terms of quantitative performance indicators, typically regarding the costs for construction, operation and also user costs and even external macroeconomic costs. Limiting values (criteria) to these indicators, however, can usually not been found. This lack of explicit limits regarding economic indicators is not surprising inasmuch the stakeholders' economic demands are hard to recognise because the general public is usually not interested in the specific budget allocation for bridge projects. Even if in some cases—well known major bridges with high maintenance expenditures—the costs may become an issue of major public interest, the stakeholders' information base is much too fuzzy to allow for a precise definition of performance criteria beyond a simple demand for a responsible allocation of public money. Also, an explicit upper limit to the owner cost (e.g. construction & maintenance) in budgetary terms can usually not be defined because the bridge authorities overall budget relates to the overall bridge stock and not to individual structures.

The only exception, where limits to economic criteria apply, is found in terms of legal regulations regarding toll fares of PPP-financed motorways (Uechtritz et al. 2005; Beckers 2005: 170–72; FStrPrivFinG 2006: §§ 3–6). However, these regulations do not provide explicit limits to the toll costs, but rather give broad indications regarding the upper bound. In addition, it must be kept in mind that a maximum upper bound regarding toll costs can also follow from competition with alternative routes (Uechtritz et al. 2005: 119–120) or from a more general level of acceptability, which can be explained by the theory of *behavioural pricing* (Gawel and Schmidt 2010: 130–135). Examples for this are the lower-than-expected numbers of users in the case of the two German PPP-projects *Warnowtunnel* and *Herrentunnel*, which both compete with toll-free alternative traffic routes (Gawel and Schmidt 2010: 83–88; Schweisfurth 2006: 24–26).

In conclusion, the definition of explicit limits, as practiced by the fib Draft Model Code 2010 for functional requirements, is not applicable to economic requirements. These should be defined as softer qualitative requirements, with verbal descriptions of the relevant economic aspect—such as service life costs—and a preferred tendency, such as 'preferably low in comparison with other bridges of this size'.

For *societal & cultural* aspects explicit performance requirements are not mentioned in the relevant literature or codes and standards—neither in terms of indicators nor in terms of limiting values or preference scales. The only exception is found in terms of the requirement for preservation of heritage structures, which can simply be defined in such a way that the lifetime of these structures shall be as long as possible. Such a preservation requirement follows, for example, directly from a listing as a UNESCO World Heritage Site or as a monument listing by national or local authorities. It can also follow indirectly from a bridge specific evaluation according to criteria that describe the worth of preservation (Tilly 2002: 9–12; Vockrodt et al. 2003: 10–15; ASTRA 1998: 8–9). Explicit requirements regarding further social impact and the aesthetic appearance, however, are not defined in the relevant literature or codes and standards. With respect to these aspects, the Draft Model Code 2010 (fib 2010b: 191–192) neither defines indicators nor associate criteria. It just recommends to measure the performance with respect to cultural, societal, and aesthetic aspects with the help of questionnaires to the affected people and relating the results to criteria that should be '*based on the regulation, the requests from an owner or* [the designer's] *intents*' (fib 2010b: 192). These criteria, however, are not specified by the Draft Model Code.

This can be explained by the fact that, in analogy to the economic aspects, both the relevance as well as the valuation of the various societal & cultural aspects is highly bridge specific. The requirement regarding the traffic capacity, for instance, certainly depends on the size and traffic importance of the bridge. Similarly, the aesthetic evaluation is expected to depend on the surroundings and the special perception of the bridge by the (local) population. In order to fill this gap, qualitative requirements should be defined with respect to societal and cultural aspects.

For the description of *environmental* aspects, many indicators are provided by the relevant literature as well as codes and standards. However, quantitative limits for the definition of requirements are usually not defined. The fib Task Group 3.6 (fib 2008b: 12–13) categorises these indicators regarding the environmental impact. The indicators of the first category describe the impact on the global environment by emissions of greenhouse gases and ozone depleting substances as well as by consumption of resources and energy. The second category refers to the regional environment in terms of pollution and contamination of air, water and soil as well as in terms of waste generation. The indicators of the third category describe the impact on the immediate site environment in terms of dust, noise and vibrations especially during construction works. And finally, the fourth group of indicators refers to the local environment, which is a closely confined surrounding, where the structure itself is influencing the health of its users—such local environment, however, usually does not apply for bridges but for buildings where harmful substances or gases can leach or escape from built-in materials. Analogous environmental aspects and indicators are also mentioned by the Draft Model Code 2010, the German Committee for Reinforced Concrete (DAfStb), and the Technical Committee 207/SC 4 of the International Organisation for Standardisation (fib 2010a: 45–46; DAfStb 2007: 25–26; DIN EN ISO 14031 (2000): 36–40).

However, the above sources do not provide explicit quantitative limits regarding these indicators. Instead, they consistently state that limiting values should be determined based on legal regulations or on preferences of the owner, the designer or the management (fib 2008b: 1; fib 2010b: 190; DIN EN ISO 14031: 5; DAfStb 2007: 16). But such criteria, as they are currently provided by codes, standards, laws and regulations often refer either to effluent standards or to the general global impact, for example overall $CO_2$ reduction goals. This follows from an overview on relevant legal specifications, which is provided by the fib bulletin 47 (fib 2008b: 20). Also, it must be taken into account that—in analogy to the economic and societal aspects—the definition of quantitative limits may have highly bridge specific facets. For example, a bridge for an electric powered railway line may be evaluated more generously in terms of $CO_2$ in comparison to a motorway bridge because it is part of a relatively climate-friendly traffic network.

In conclusion, the following definition for the term *owner's (performance) requirement*s is used in the scope of this thesis.

**Definition 11**: The term **(owner's) performance requirements** stands for a set of specifications that describe the characteristics and the behaviour (=performance) of the bridge which is necessary to fulfil the demands of the stakeholders. They are either specified in quantitative or qualitative form. In quantitative form they are expressed in terms of a performance indicator—a parameter that quantitatively describes a performance aspect—and associated performance criterion, which in terms of quantitative limits defines the border between the desired and adverse behaviour. In their qualitative form they describe verbally (or in terms of a numeric indicator) the performance aspect and the preferred tendency of these aspects. Like the stakeholders' demands they refer to functional, economic, societal & cultural, and environmental aspects. Functional requirements are usually expressed in quantitative form, whereas the other requirements usually can only be described qualitatively.

### 2.5.4 Objectives in Bridge Design and Maintenance

By following the logical sequence of developing performance requirements from the stakeholders' demands, the next step is the final translation of the performance requirements into explicitly defined *maintenance* and *design objectives.*

The *design objectives* are to be accomplished as a result of the design process. They can, for instance, be derived from the ten characteristics of Fritz Leonhardt (Leonhardt 1984: 26–30), which specifically refer to the aesthetic quality of bridges. However, in this thesis design objectives are not dealt with in detail because the actual investigation focuses on the in-service assessment and maintenance of bridges.

The *maintenance objectives* are to be accomplished as a result of the applied maintenance strategy. The European Standard EN 13306 ('Maintenance – maintenance terminology'), for example, defines the term maintenance objective as a *'target assigned and accepted for the maintenance activities.* (…) *These targets may include for example availability, cost reduction, product quality, environment preservation, safety, asset value preservation*' (DIN EN 13306: 6–7). Together with other authors, this standard names a consistent set of quite general objectives that should be accomplished by maintenance planning (EN 13306: 4; fib 2010a. 48; fib 2010b: 260; fib 2002: 3; Haardt 2002: 9–10). These objectives can be categorised into the same groups as the stakeholders' demands and the owner's performance requirements: function, economy, society & culture, environment:

- minimise *costs* (economy)
- satisfy *functional* requirements, such as safety and serviceability (function)
- satisfy *service life*, *durability* and *preservation* requirements (function, society & culture)
- satisfy overall bridge *condition* requirements (function, society & culture)
- satisfy *availability* requirements (function, society & culture)
- satisfy *societal & cultural* requirements, e.g. *aesthetic appearance* (society & culture)
- satisfy *environmental* requirements (environment)

It can be noticed that the economic objective is clearly defined as minimisation of costs, whereas all other objectives refer to the satisfaction of indeterminate requirements. The above sources differ in their described approaches to this goal: The FIP Commission 10 and Haardt both refer to the compliance with limiting values whereas the Draft Model Code 2010 refers to multi-objective optimisation, which '*minimise*[s] *the expected costs on a net present value basis* (…) *in conjunction with minimising adverse environmental and social impacts*' (fib 2010a. 48). However, the Draft Model Code 2010 acknowledges that

> '*In contemporary engineering practice a practical approach is to minimise the costs associated with achieving the required performance (i.e. to meet relevant performance criteria during the service life at the required reliability level) whilst achieving an appropriate (minimum) quality requirement.*' (fib 2010a: 48)

In order to support an object oriented maintenance planning and to allow for a comprehensible evaluation of maintenance strategies, the objectives should be expressed in measurable form. The performance indicators as well as their preferred tendency and limiting values must be developed in consistency with the underlying performance requirements and the derived maintenance objectives. Because bridge maintenance planning has a long term perspective, a suitable indicator must condense the service-life performance to a single numerical value. Also, it must be sufficiently sensitive in order to reveal the effects of alternative maintenance strategies on the service-life performance. The following Table 2.2 lists a set of example maintenance objectives including a general description of example performance indicators and associate criteria.

**Table 2.2:** Maintenance objectives (examples)

| Objective | Indicator | Example criterion |
|---|---|---|
| *Functional* | | |
| Good overall condition | Moving average of the *CI* [a] | > 2.0 as 10-year average [b] |
| *Economic* | | |
| Low life cycle costs | Net present value of costs | Preferably low |
| Stabilised expenditures | Moving average of expenditures | Stable 10-year average |
| *Societal & cultural* | | |
| Heritage preservation | Service life | Maximise |
| Acceptable appearance | Time with visible deterioration [c] | Preferably short |
| Good traffic availability | Average traffic availability [d] | Preferably high |
| *Environmental* | | |
| Low greenhouse effect | $CO_2$-equivalent emissions | Preferably low |
| Low resource use | Pavement renewal interventions | Rarely |

[a] *CI*: condition index

[b] Scale according to German RI-EBW-PRÜF (2007, 2013): 1.0–1.4 (very good), 1.5–1.9 (good), 2.0–2.4 (fair), 2.5–2.9 (acceptable), 3.0–3.4 (bad), 3.5–4.0 (critical)

[c] An aesthetic threshold must be defined (e.g. occurrence of noticeable cracks and spalling).

[d] Availability: ratio of actual traffic capacity to the design traffic capacity. Beneath its societal implications it is also a *functional* issue.

In conclusion, the following definition for the term *maintenance objective* is used in the scope of this thesis.

> **Definition 12**: The term **maintenance objective** stands for a target assigned to maintenance planning. It is expressed in terms of a performance indicator and an associate criterion. The performance indicator must quantitatively represent a service-life performance aspect in such a way that it plausibly condenses all relevant facets and it must be sufficiently sensitive to reveal the effects of alternative maintenance strategies on the service-life performance. The criterion is usually expressed in terms of a preferred tendency.

### 2.5.5 Preference Profiles

A central assumption in this thesis, which is empirically supported by the investigation in chapter 3, is that bridges differ with respect to the stakeholders', owner's and bridge manager's preferences regarding the various performance aspects. It is assumed that the relative valuations of the aspects depend on the special situation of the bridge, such as the surroundings and the traffic importance. The term *preference profile* is used in this thesis for describing these bridge specific combinations of aspect valuations.

These preference profiles are illustrated in terms of diagrams that are suitable to reveal characteristic feature of the bridge. In this thesis, the profiles are illustrated in the form of so-called *star plots* (also known as *spider chart* or *radar diagrams*). This type of diagram displays multivariate data as data-points on axes which originate circular and which are equally spaced from one origin. The data-points are connected by a polygonal line resulting in a star-shaped plot. Star plots are preferably used to highlight or compare dominant variables as well as relative values of the multivariate data groups and to reveal similarities between such data groups (Chambers et al. 1983).

The following Fig. 2.6 shows the example preference profiles of the owner's requirements ratings regarding the aspects *traffic availability*, *service life*, *aesthetic appearance* and *costs* for two example bridges:

- *Bridge 1*: Low importance of traffic availability, very high importance of service life and appearance (e.g. historic monumental bridge), relatively low importance of maintenance costs (because it is worth to spend extra money for preservation)
- *Bridge 2*: Very high importance of traffic availability (e.g. motorway bridge), high importance of service life and costs, relatively low importance of appearance (e.g. located in a remote area)

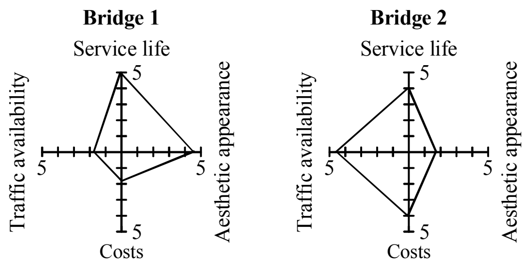



**Fig. 2.6:** Example star plots with the preference profiles of bridge 1 and 2

In conclusion, the following definition for the term *preference profile* is used in the scope of this thesis.

> **Definition 13**: The term **preference profile** describes the bridge specific combination of valuations regarding the various performance aspects such as traffic availability, service life, aesthetic appearance and costs. Preference profiles can be formulated on three stages—stakeholders' demands, owner's requirements, and maintenance objectives—in the form of demand profiles, requirement profiles, and objective profiles. In this thesis they are illustrated in the form of star plots in order to highlight dominant preferences and characteristic valuation biases.

### 2.5.6 Development Phases of Requirements and Objectives

The Draft Model Code 2010 describes the process of defining the performance requirements by the term *requirements development phase*. This '*phase of extracting and describing performance requirements*' is described as being subdivided into four consecutive steps (fib 2010a: 20):

- gathering the stakeholders' demands
- checking the findings (demands) for consistency and completeness
- definition of requirements in descriptive terms
- specification of performance requirements in terms of indicators and criteria

Analogously, the *objectives development phase* regarding the design and maintenance objectives can be subdivided into the four following steps:

- gathering the performance requirements
- definition of maintenance/design objectives in descriptive terms
- specification of objectives in terms of indicators and criteria

Both processes are common engineering practice during the design of new bridges. For example the German regulation for determining the fees for engineering and architectural services—the *HOAI*—describes the detailed scope of actions within all consecutive design and execution phases of objects like buildings, infrastructure structures and landscaping. There, the first five phases—*establishing the basis of the project*, *preliminary design, final design*, *building permission application*, *execution drawings*—contain activities that clearly describe a requirements development phase with inclusion of stakeholders and subsequent processes of deriving design objectives from those findings (HOAI 2009, 2013: Anlage 12; HOAI 2004: 222). Also the German legal regulations regarding the public building planning law (e.g. BauGB 2011: § 3) as well as regarding the planning approval procedures (VwVfG 2009: §§ 72–78; VerkPBG 2006: § 3) implement the consideration of the various stakeholders' demands for the definition of requirements and formulation of design objectives.

In contrast to the above described bridge design phase, no regulated procedures are found that describe an analogous development of objectives during the in-service maintenance phase. The above mentioned German regulations—*HOAI*, *BauGB*, *VwVfG* and *VerkPBG*—do not apply to the maintenance planning of bridge structures. Also the German collective guideline for the maintenance of civil engineering structures RI-ERH-ING (2007), which encompasses five separate guidelines, with two of them referring to the planning and evaluation of bridge maintenance measures (RPE-ING 2012 and RI-WI-BRÜ 2004), does not describe objective development phases for maintenance objectives. Instead, the RPE-ING—a description of the new German bridge management system *BMS*—provides a set of predefined objectives for maintenance strategies:

- *individual bridge*: minimise the total costs and reaching an acceptable level of the condition, traffic capacity, safety, and environmental impact
- *bridge stock*: optimise overall condition of all bridges at given budget or minimise necessary budget while maintaining a minimum overall condition of all bridges

The guideline RI-WI-BRÜ (2004)—a guideline for the comparative evaluation of major intervention alternatives—puts a strong focus on owner costs but also considers other aspects (function, traffic capacity, environment, general feasibility) in qualitative form. However, the valuation of these aspects is left to personal judgement.

A further example is the UK Department for Transport code of practice *Management of Highway* Structure. It contains a full chapter on '*stakeholder consultation and involvement*' (DfT 2005: 68–75), but this chapter provides guidance rather for the execution phase of maintenance than for the strategic planning of interventions and strategies.

In conclusion, regulated procedures for deriving requirements and objectives from the stakeholders' demands are only found with respect to the design of new bridges. The definition of maintenance objectives, however, seems to be left to individual implementation by the individual bridge authorities and the responsible bridge managers.

## 2.6 Bridge Maintenance

### 2.6.1 Bridge Maintenance, Maintenance Activities, and Maintenance Planning

A general definition of the term *maintenance* is provided by the European Standard EN 13306 (2010). This standard provides an equivalent maintenance terminology in three European languages (English, German and French) and defines maintenance as a '*combination of all technical, administrative and managerial actions during the life cycle of an item intended to retain it in, or restore it to a state in which it can perform the required function.*' (DIN EN 13306: 6).

In the field of bridge engineering, however, the term maintenance is used inconsistently. On the one hand some *fib*-publications (fib 2002: 98; fib 2008b: 123; fib 2010a: 14) use the term *maintenance* for exclusively describing activities that are described as *routine minor maintenance* activities ('*Wartung*') by the DIN EN 13306 as opposed to repair and rehabilitation. On the other hand, these *fib*-publications use the terms *intervention* (fib 2008b: 123) or *conservation* (fib 2010a: 27) for those activities that are referred to by the term maintenance by the DIN EN 13306. On the other hand, Wallbank et al. (1999: 164) and Denton (2004: 3) use the term maintenance as a collective term, which encompasses all activities for retaining or restore a bridge to a state where it can perform as required. Also the *fib*-publications on concrete structure management in this connection occasionally also use the term maintenance (fib 2002: 16) in the same meaning as the DIN EN 13306.

In the scope of this thesis, the term bridge maintenance is defined on the basis of the definition that is used by the DIN EN 13306:

> **Definition 14**: The term **(bridge) maintenance** describes the combination of all technical, administrative and managerial actions during the life cycle of a bridge intended to retain it in, or restore it to a state in which it can perform as it is required.

As a further step, the terms for describing and categorising maintenance activities are defined in the scope of the present thesis on the basis of the relevant literature. This is done with strong reference to the German standard DIN 31051, which is in use parallel to the DIN EN 13306. In contrast to the DIN EN 13306, which only serves as an aid to translation in connection with business transactions (DIN 31051: 1, 11), it provides a structuring of all maintenance actions into the four basic activities *Inspektion*, *Wartung*, *Instandsetzung*, and *Verbesserung*, which can be translated to *inspection* (including assessment and evaluation), *routine minor maintenance*, *repair*, and *improvement* (DIN 31051: 2). In summary, these basic activities can be described as follows:

- *Inspection*: Gathering, interpreting, processing, representing and valuating the data on the bridge condition in terms of conformity with the requirements (DIN EN 13306: *inspection*, *monitoring*, *compliance test*, *function check out*, *fault diagnosis*, *fault location*)
- *Routine minor maintenance*: Regular or repeated simple activities such as cleaning (DIN EN 13306: *routine maintenance*)
- *Repair*: Activities applied in order to restore the required function (DIN EN 13306: *overhaul*, *restoration*, *repair*, *temporary repair*)
- *Improvement*: Activities that ameliorate the reliability, maintainability or safety without changing the function of the bridge (DIN EN 13306: *improvement*)

This definition of the basic maintenance activities is also applied in the scope of this thesis. However, some adjustments of these definitions are necessary in order to account for bridge specific particularities. First of all, the present thesis clearly separates the three activities inspection, assessment and evaluation from maintenance. This is in order to separate passive information gathering from active planning and maintenance execution. An analogous separation is followed by the relevant literature that deals with bridge management, inspection and maintenance (e.g. fib 2008b, fib 2002, Ponnuswamy 2009: 543–593). Second, the additional basic activity *special preventive maintenance* is introduced. This activity plays a particular role in bridge engineering and is clearly distinct from routine minor maintenance on the one hand and repair activities on the other hand.

Following this review of the relevant literature, *bridge maintenance* is subdivided into four basic activities: *routine minor maintenance*, *special preventive maintenance, repair / replacement*, and *improvement*. Although these basic activities each include technical, administrative and managerial aspects (e.g. DIN 31051: 3–5), in the present thesis the focus is set on the technical aspects. These are seen as the distinguishing factor, while the administrative and managerial issues concern the operational accomplishment of these activities. The distinguishing feature in this case is, how the activities affect the performance of the bridge—if they are preventing or delaying deterioration, if they are responding to actual damages, or if they are even improving the actual performance level. Consequently the four basic maintenance activities are defined as follows:

*Routine minor maintenance* activities are very simple activities, which are regularly applied in order to maintain the availability for traffic and to compensate for adverse effects of traffic and weather on the structure (Wallbank et al. 1999: 164; Henriksen 2000: I-5 / 1).

*Special preventive maintenance*—also labelled *preventative* (e.g. Denton 2004) or *proactive* (e.g. fib 2010b)—is applied in order to prevent, reduce or delay future deterioration and failures. In the scope of this thesis, only activities whose *special* purpose is to prevent or delay deterioration are allocated to this category. Examples are the application of concrete surface treatments and protective steelwork repainting, which both are effective measures against steel corrosion. In contrast to this, activities like the cyclical replacement of expansion joints are not allocated to this category. This is because expansion joints serve a clearly defined structural purpose, which is absorbing the heat-induced expansions of the bridge deck. Therefore, their early replacement can rather be described as proactive bridge equipment repair—even if such early repair works also result in reduced reinforcement corrosion in the vicinity of expansion joints.

*Repair* and *replacement* activities are remedial or corrective measures, which are applied in order to re-establish a satisfying and functional state or to arrest active and already advanced deterioration (fib 2010b: 263). These measures are typically applied after damage has already occurred. Some authors refer to this type of measures by different terms, such as '*reactive*', '*corrective*' or '*essential*' maintenance activities (fib 2010b: 261–263; Wallbank et al. 1999: 164; Denton 2004: 3). However, irrespective of different labelling, there is broad agreement on the definition of this type of maintenance activity. In the scope of this thesis activities like rebuilding or replacement of bridge elements are subsumed under the terms *repair* and *replacement*. Thus, typical repair activities are *concrete repair* (e.g. sealing of crack), *pavement resurfacing* and *re-waterproofing*, and *replacement of expansion joints*.

The *improvement* activities are applied in order to upgrade the actual bridge performance, for example the strength, without changing the function (as opposed to *modification*). Thus, typical improvement activities are measures for structural strengthening such as adding reinforcement (external embedment or in drilled holes), adding concrete (shotcreting) or external prestressing.

In summary, the following definition for the term *maintenance activities* is used in the scope of this thesis.

> **Definition 15**: The term **maintenance activity** describes an activity that is conducted in order to retain a bridge in a state in which it can perform the required function or, if necessary, restore it to such a state. In general, maintenance activities can be categorised into four groups: routine minor maintenance, special preventive activities, repair, and improvement.

The term *maintenance planning* is neither explicitly defined by the standards DIN EN 13306 and DIN 31051 nor by the above mentioned *fib*-publications or by Wallbank et al. and Denton. Therefore, in the scope of this thesis, maintenance planning is defined as follows:

> **Definition 16**: The term **maintenance planning** describes the object-oriented process of formulating a long term plan for the application of maintenance activities, which promises that all performance requirements will be fulfilled. In general the planning process encompasses the following four steps: (1) identifying the performance requirements and formulating the maintenance objectives, (2) defining and assessing alternative maintenance strategies, (3) evaluating the alternative strategies, and finally (4) selecting the most convenient strategy and formulating the long term maintenance plan.

### 2.6.2 Maintenance Strategies

The literature that deals with bridge maintenance management usually uses the term *maintenance strategy* for describing a specific combination of maintenance actions and their application parameters, such as time of first application and cycle-time or combination rules (e.g. Neves and Frangopol (2005), Petcherdchoo et al. (2008), Denton (2004), Frangopol and Estes (1997)). On the one hand, these definitions are very descriptive in terms of the technical aspects, but one the other hand they do not refer to the underlying process of planning and decision taking. In other words, these definitions describe the outcome rather than the way how the decisions have been made.

A more general definition for the term maintenance strategy is used by the DIN EN 13306, which defines it as a '*Management method used in order to achieve the maintenance objectives.*' (p. 9). A similar definition is offered by Ryall (2010), who defines the term strategy as '(…) *a method for making, doing or accomplishing something. It considers plans in the long term.*' (p. 363). This emphasis of the objectives in the definitions of the term strategy is also found in the work of Vollrath and Tathoff (2002: 23–35).

The present thesis uses an analogous definition, which puts emphasis on the general methods and decision rules for attaining defined combinations of maintenance objectives. The following definition of the term *maintenance strategy* is used within the scope of this thesis:

**Definition 17**: The term **maintenance strategy** describes the underlying concept of the object-oriented, holistic and long-term-oriented selection, combination and application of maintenance activities.

### 2.6.3 Classification of Bridge Maintenance Strategies

**Literature Review**

The relevant literature usually classifies maintenance strategies both on the basis of the applied maintenance activities and on the basis of the deterioration level at which maintenance activities are applied. Typical distinctions in this context refer to the preventive, proactive and reactive characteristics. Less common strategy distinctions refer to a possibly targeted performance-development or specific maintenance application regimes.

The *preventive characteristics* of maintenance strategies are determined by the composition of the activities package. Generally, a high significance of special preventive activities and also a high-intensity application of routine maintenance both characterise a strategy with *preventive bias*.

The *proactive/reactive characteristics* are determined by the deterioration level at which maintenance activities are regularly applied. A perfectly proactive maintenance application is defined as action taking before the onset of (noticeable) deterioration, whereas a perfectly reactive application is defined as the repair or replacement of failed items (Yang et al. 2006: 200; fib 2010b: 261–263). The overall *proactive* or *reactive bias* of a maintenance strategy describes the relative weight of proactive and reactive maintenance interventions.

The *targeted performance level development* is used by some authors as a descriptive characteristic of maintenance strategies. This aspect distinguishes if the strategy aims at maintaining a certain performance level, or if it aims at improving the bridge performance or if it even allows for a controlled performance decrease. However, this characteristic is rarely used for bridge specific strategy differentiation. The Draft Model Code 2010, for example, uses this aspect only in order to characterise maintenance activities (fib 2010b: 261) but not strategies. Vassie and Arya (2006: 83–84) and the UK Department for Transportation (DfT 2005: 135) use the terms *steady-state*, *enhancement*, and *managed deterioration* for describing strategies that pursue the above mentioned three performance developments. In this context, steady-state maintenance aims at stabilising the overall performance at a certain level, while the enhancement approach pursues to enhance the performance to a higher level in the long run. However, these authors use the two strategy variants exclusively for bridge stock management and not for individual bridges. A steady-state strategy referring explicitly to the overall bridge stock is also pursued by the Düsseldorf bridge authority in Germany (Vollrath and Tathoff 2002: 23–25). Only the managed deterioration strategy is mentioned in connection with individual structures '*if decommissioning or replacement is planned in the near future*' (DfT 2005: 135). A special variant of this strategy is the *no maintenance / replacement* (Vassie and

Arya 2006: 83) or the *do nothing and rebuild* (Neves and Frangopol 2005: 191–193) strategy, which does without any maintenance and finally results in a replacement of the whole structure.

The *application regime* describes a further characteristic of maintenance strategies (fib 2010b: 260–263). It differentiates between a *condition based regime*, where the maintenance activities are conducted when the condition reaches a defined threshold, and a *time based regime*, where the activities are conducted according to a fixed time-schedule. However, in the relevant literature this characteristic is not used for a principal strategy differentiation as it is the case with the preventive and proactive/reactive characteristics. Rather it is seen as a variation on the *tactical* level (fib 2010b: 262–263), which allows for detailed definitions, for example in the form of 'preventive strategy with condition based maintenance application'.

**Strategy-Classification in the Scope of the Thesis**

In scope of this thesis, maintenance strategies are classified—according to common practice—on the basis of their preventive as well as their proactive/reactive characteristics. The application regime (condition- or time-based) is only used for describing variations at the tactical level. The targeted performance level development (e.g. steady-state, enhancement, managed deterioration) is not used as a characteristic for strategy differentiation in the scope of this thesis. This is because the performance level development rather describes a maintenance objective (see Definition 12) than a maintenance strategy (see Definition 17), which is defined as the underlying concept of maintenance planning.

Based on these characteristics, seven basic strategy types are defined and described in terms of their individual underlying concepts of activity selection, combination and application:

- full preventive strategy
- reactive repair strategy
- proactive repair strategy
- preventive repair strategy
- preventive-proactive strategy
- selective strategies
- do nothing & rebuild

The *full preventive strategy* extensively applies special preventive measures with the objective to conserve the current structural condition by either completely preventing deterioration from initiating and taking place at all or by slowing it down considerably. Another characteristic of this strategy type is the intensive application of routine minor maintenance. In its pure-form this strategy aims at precluding any repair and strengthening interventions during the planned service life (Vassie and Arya 2006: 83).

However, in practice this pure-form is rarely applied. One reason for this can be found in the traffic disruptions that potentially arise in connection with the preventive activities (Vassie and Arya 2006: 83). Another reason may be found in limited maintenance budgets that lead to the sacrifice of not urgent preventive activities for the sake of necessary repair measures. Moreover, the complete preclusion of any repair activities by special preventive measures is usually not feasible on technical grounds so that repair activities cannot completely be precluded.

The *reactive repair strategy*—also known by the name *essential strategy* (Vassie and Arya 2006: 83, Yang et al. 2006: 200)—applies repair and strengthening activities only when they become urgently necessary because of an actual or imminent violation of performance requirements. This can be the case with damages that already cause traffic disturbances or defects that constitute clear and present threats to structural safety. The objective of this strategy is to minimise the number of interventions and to save maintenance expenditures in the short run. Naturally, this strategy follows a condition based application regime as it reacts to critical condition developments. The special feature of this strategy is that the condition threshold for maintenance application is the point at which the performance requirements are actually violated or their violation is imminent.

This kind of strategy is often applied in practice. However, the actual difference between this strategy and the proactive repair strategy, which is introduced below, is not always clear. This is because it is only a question of the deterioration level at which maintenance activities are applied.

The *proactive repair strategy* is closely related to the above described reactive repair strategy. This strategy aims at maintaining a continuously good performance and also at precluding major repair interventions by applying repair and strengthening measures timely before an actual or imminent violation of performance requirements occurs. Naturally, this strategy also follows a condition based application regime as it reacts to condition developments. The special feature of this strategy is that the condition threshold for maintenance application is set at an uncritical level in order to intervene very early and thus avoid major repairs in the long run.

This kind of strategy is also regularly applied in maintenance practice. However, as it is mentioned above, the practical differentiation of this strategy from the reactive repair strategy is not always clear.

The *preventive repair strategy* is a hybrid strategy, which combines a strong preventive bias with reactive repair activities. On the one hand, this strategy differs from the reactive repair strategy because of the application of special preventive measures and intensive routine minor maintenance. On the other hand, it differs from the fully preventive approach inasmuch as the special preventive activities are applied on a less strict basis. For example, they may only be applied during erection in order to postpone the time of deterioration initiation—thereafter the preventive measures may not be repeated—or they may be applied only in longer time intervals so that repair activities become necessary. The objective of this strategy is to postpone repair and strengthening interventions by preventive activities—in contrast to the full preventive approach, which aims at preventing deterioration at all.

This strategy is applied in maintenance practice as a pragmatic version of the full preventive strategy, which is described above. The objective of postponing necessary repair and strengthening interventions is far more realistic than trying to preclude them altogether. The strategy is closely related to the preventive-proactive repair strategy, which is introduced below. Because the difference is only the set threshold for maintenance application, a differentiation is not always clear.

The *preventive-proactive strategy* is also a hybrid strategy, which combines special preventive activities and intense routine minor maintenance with proactive repair activities. The objective of this strategy is similar to the preventive repair strategy—only that in addition to postponing repair activities it also aims at precluding major interventions and at maintaining a continuously good performance.

The repair and strengthening interventions are conducted according to a condition based application regime with the threshold for maintenance application being set at an uncritical level in order to intervene very early and thus avoid major repairs in the long run. The preventive activities are conducted in analogy to the above described preventive repair strategy, allowing for deterioration to take place between repeated preventive maintenance applications.

This strategy is also applied by bridge authorities as a practical version of the full preventive strategy. The objective of postponing repair and strengthening and also avoiding major interventions is far more realistic than trying to preclude them altogether.

The term *selective strategy,* in the scope of this thesis, refers to strategies that show specific preventive and proactive/reactive characteristics with respect to individual bridge elements. An example for this is the preventive regular repainting of steel elements in connection with no application of preventive activities regarding the concrete elements and only regular-interval routine maintenance.

The *do nothing & rebuild strategy* does without any maintenance and finally results in a replacement of the whole structure. For example, this strategy may be applied if a bridge is affected by higher level development planning, such as the widening of motorways. In this case, maintenance may be deferred because the bridge will be replaced together with the future works anyway.

The following Table 2.3 summarises the descriptions of the seven strategy types by highlighting the specific maintenance objectives and the underlying maintenance applications concepts.

**Table 2.3:** Types of maintenance strategies

| Strategy | Objective | Underlying concept |
|---|---|---|
| *Pure forms* | | |
| Full preventive | Preclude any repair and strengthening interventions | High intensity routine minor maintenance and extensive application of special preventive measures so that all potential deterioration processes are either prevented or slowed down considerably |
| (reactive) Repair | Minimise the number of interventions (in the short run) | React only to critical developments by urgent repair and strengthening activities.<br>No preventive maintenance |
| Proactive repair | Maintain a continuously good performance<br>Preclude major repair / strengthening interventions | Foresighted early application of repair and strengthening at low deterioration levels.<br>No preventive maintenance |
| Do nothing & rebuild | No interventions at all | No interventions at all |
| *Hybrid forms* | | |
| Preventive repair | Postpone repair and strengthening interventions | Thorough routine minor maintenance and application of special preventive measures.<br>React only to critical developments by urgent repair and strengthening activities |
| Preventive-proactive | Maintain a continuously good performance<br>Postpone repair and strengthening interventions<br>Preclude major interventions | Thorough routine minor maintenance and application of special preventive measures<br>Foresighted early application of repair and strengthening at low deterioration levels |
| Selective | Diverse picture of preventive and pro-/reactive characteristics for individual bridge elements | Diverse picture of the above approaches for individual bridge elements |

The characteristic features of the seven strategy types are be depicted in terms of *strategy profiles* with the help of star plots similar to the preference profiles (section 2.5). The following Fig. 2.7 shows the example strategy profiles with respect to the four aspects *intensity of preventive activities*, *intensity of routine maintenance, deterioration level of structural elements,* and *deterioration level of bridge equipment.*

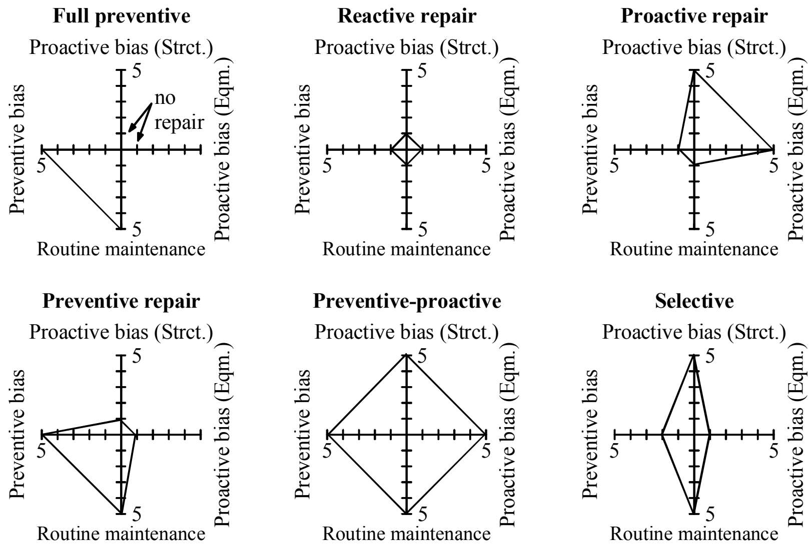



**Fig. 2.7:** Example star plots with strategy profiles for basic strategy types as they are described above (Strct.: Structural; Eqm.: Bridge equipment)

## 2.7 Assessment of Maintenance Strategies

### 2.7.1 Overview

The term *strategy assessment* describes analytical investigations, which indicate and predict the bridge performance as a result of the applied strategy. In analogy to the linked terms ‘bridge assessment’ and ‘bridge evaluation’ (see section 2.4), strategy assessment provides the performance-data, which are valuated with respect to the maintenance objectives during the subsequent strategy evaluation in order to find out the most promising maintenance strategy.

Strategy assessment must be conducted with respect to those performance aspects, which are important to the stakeholders and the owners of a bridge, as it is explained in the preceding section 2.5: functional, economic, societal & cultural, and environmental. Accordingly, the indicators that are used for performance assessment must agree with the indicators used as part of the maintenance objectives (Definition 12) in order to allow for strategy evaluation with respect to the objective profile (Definition 13). Furthermore, the assessment indicators must condense the performance development over a time horizon, for example over the planned service life, in a single numerical value. This is necessary because strategic bridge maintenance planning refers to a long time horizon and thus strategies must be assessed accordingly. Such a condensation can be achieved, for example, by averaging the performance development over the relevant time interval.

It is found by a review of the relevant literature that the traditional practice of strategy assessment focuses on structural safety, overall bridge condition and the economic performance. For these aspects long-known assessment procedures and indicators are routinely applied and commonly agreed upon. In addition to this, new aspects have been introduced over the last years: The upcoming sustainability debate shifts the focus from the classic functional and economic aspects to the broader and less defined fields of societal, cultural and environmental aspects.

Societal aspects have become a common issue in strategy assessment—even if this usually does not go further than considering user costs due to traffic delays from maintenance activities (e.g. Denton 2004, Pommerening et al. 2008). However, further societal and cultural aspects, such as aesthetical value or heritage preservation, have only exceptionally been found to be applied in actual assessment of maintenance strategies (Branco and de Brito 2004)—although in the relevant literature these two issues are widely mentioned as being potentially important as requirements.

Also, in connection with the more and more alarming worldwide development, environmental aspects are increasingly considered to be important for the assessment of bridge maintenance strategies. With respect to these environmental aspects, sophisticated assessment procedures and indicators are provided by the literature. However, it is found that until now, environmental maintenance strategy assessment is usually conducted by calculating a monetarised environmental impact—a method which is not uncontested because of inconsistent monetary evaluation of environmental impacts and possibly significant dimensional differences compared to real owner costs.

In consideration of these debates, the present thesis will develop and propose an assessment approach where all relevant aspects—from the functional, the economic, the societal/cultural, and the environmental domain—are assessed separately in terms of appropriate indicators and undistorted by monetarisation. In analogy to the preference profiles (Definition 13), a *strategy performance profile* will be used for the overall strategy assessment. Only by means of such a differentiated strategy performance profile assessment it is possible to provide the detailed data that are necessary for the bridge specific strategy evaluation that accounts for the individual preference profiles of each bridge.

The following definitions will be used in the scope of this thesis:

- **Definition 18**: The term **strategy performance profile** describes the bridge specific performance regarding the various aspects as a result of the (hypothetically) applied maintenance strategy. Crucially, the indicators for describing the performance aspects represent the performance development over an explicitly defined time period in a single numeric value. It can be useful to illustrate the profiles in the form of star plots in order to highlight characteristic performance results of the strategy.
- **Definition 19**: The term **strategy assessment** describes the overall process of calculating and illustrating the strategy performance profiles. The overall strategy assessment process encompasses three assessment activities: (1) defining a set of representative performance indicators, (2) defining a theoretical model of the bridge that allows the calculation of the performance profile, (3) simulation of the performance development and representing the results in terms of a service life performance profile. Strategy assessment is clearly distinguished from strategy evaluation as it exclusively indicates and predicts the performance as a result of the applied strategy but does not valuate the resulting performance profile with respect to the maintenance objective profile.

### 2.7.2 Strategy Assessment with Respect to Bridge Condition

The literature review shows that strategy assessment with respect to the *bridge condition* is achieved by two different approaches. The first one estimates the development of the overall bridge condition index (*CI*) in a simplified way. The second approach conducts a more detailed assessment with respect to individual element condition indices ($CI_{elem}$), such as the condition of the asphalt pavement, of the cross-beams and of the main girders. A resulting overall bridge condition index (*CI*) is calculated based on these individual elements.

The first simplifying approach usually assumes a linear deterioration of an overall condition index (e.g. Neves and Frangopol 2005, Neves 2005, Denton 2004):

$$CI = CI_0 - \alpha\left(t - t_0\right) \tag{2.1}$$

with

| | |
|---|---|
| $CI$ | overall bridge condition index |
| $CI_0$ | initial condition index |
| $\alpha$ | condition deterioration rate, the sign depends on the $CI$-scale (e.g. negative sign if $CI$=1 is 'good' and $CI$=4 is 'bad') |
| $t$, $t_0$ | time, starting time ($t_0$) |

However, this simplifying approach does not provide a differentiate picture of the condition development of individual bridge elements, such as the pavement, the bearings, or the deck slab. Moreover, modelling of the maintenance effects on the bridge condition can be rather opaque because the maintenance effects must be described with respect to the overall condition index, even if the maintenance activity affects just individual elements (e.g. Denton 2004: 47–50).

This approach is preferably used within investigations that refer to whole bridge stocks (e.g. Denton 2004) or to large groups of bridge elements, such as concrete crossheads (e.g. Neves and Frangopol 2005, Neves 2005), or bridge girders and deck slabs (e.g. Petcherdchoo et al. 2008). However, due to its very general character this approach is less suitable for detailed investigations regarding maintenance strategies for individual bridges.

The second more differentiated approach, which for example is applied by Pommerening et al. (2008), calculates the overall condition index from the condition indices of individual bridge elements. The development of the individual element condition can, for example, be assumed to be linear (equation 2.2) or exponential (see equation 2.3). The overall condition index is usually calculated based on the elements' condition by two alternative methods, which are also applied for bridge condition assessment (see 2.4.4): worst element classification (equation 2.4) and weighted condition rating (equation 2.5).

Element condition:

$$CI_{Elem,i} = CI_{0,Elem,i} - \alpha_{Elem,i}\left(t - t_0\right) \tag{2.2}$$

$$CI_{Elem,i} = m \times CI_{0,Elem,i} - n \times CI_{0,Elem,i} e^{(t/k)^p} \qquad \text{for} \qquad 0 \le t \le T_{SL} \tag{2.3}$$

The *worst element classification* depends on the $CI$-scale:

$$CI = \max_i \left(CI_{Elem,i}\right) \quad \text{or alternatively} \quad CI = \min_i \left(CI_{Elem,i}\right) \tag{2.4}$$

($CI = \max_j \left(CI_{Elem,j}\right)$ if $CI$=1 is 'good' and $CI$=4 is 'bad')

The *weighted condition rating*:

$$CI = \sum_{i=1}^{N_{Elem}} w_i \times CI_{Elem,i} \tag{2.5}$$

with

| | |
|---|---|
| $N_{Elem}$ | number of elements (e.g. 'girders', 'abutments'...) |
| $w_i$ | weighting factor of element '$i$' $\left(\sum w_i = 1\right)$ |
| $CI_{Elem,i}$ | condition index of the i$^{th}$ element |
| $CI_{0,Elem,i}$ | initial condition index of the i$^{th}$ element |
| $\alpha_{Elem,i}$ | element condition deterioration rate, the sign depends on the $CI$-scale (e.g. negative if $CI$=1 is 'good' and $CI$=4 is 'bad') |
| $t, t_0$ | time, starting time ($t_0$) |
| $T_{SL}$ | service life |
| $m, n, p, k$ | coefficients |

This element-wise approach is suitable for a detailed strategy assessment. Thus it is usually applied for bridge specific maintenance strategy assessment—for example by Pommerening et al. (2008: 15–28, 51–65), who apply a worst element condition rating, and the DAfStb (2007: 92–94), which proposes a simple weighted condition rating for maintenance strategy assessment. This more sophisticated approach provides a differentiated assessment of maintenance effects on individual bridge elements and their related implications on the overall condition. Consequently, the main field of this approach is the strategy assessment for individual bridges that takes into account detailed aspects of the structural configuration, of the requirements and of the maintenance objectives.

### 2.7.3 Strategy Assessment with Respect to Structural Safety

#### Overview

Strategy assessment with respect to the *structural safety* is usually achieved by two different approaches. The first *deterministic code-based approach* calculates and compares the loads and the strength of structural members based on relevant standards and codes of practice. It assesses how well the structural behaviour of the bridge, as a consequence to the applied strategy, complies with safety requirements as they are defined by standards and codes of practice. The second *probabilistic approach* calculates the structural reliability, which is complementary to the probability of failure, as a measure of the structural safety (JCSS 2001: 14). Thus, this approach assesses how well the bridge performs with respect to codified requirements regarding the reliability level (e.g. fib 2010a: 38–42).

Either way, the structural safety changes with time because of influences such as deterioration mechanisms, increasing traffic loads, and maintenance activities. According to the relevant literature, the time-depend development of the structural safety can be estimated by two methods. On the one hand, a simplifying assessment of the overall structural safety, which applies a linear expression, is suitable for strategy assessment with respect to bridge stocks. On the other hand a detailed repeated calculation of the structural safety is suitable for the detailed strategy assessment with respect to individual bridges.

#### Deterministic Approach

According to the *deterministic code-based approach* the structural safety is preferably expressed in terms of the ratio $\eta$ of the strength ($R$) to loading ($S$), which both are calculated based on relevant standards and codes of practice, such as the DIN FB 101, DIN FB 102, BMVBS 2011, or the fib 2010a/b. This is a very practicable approach, which applies commonly accepted and consistent methodologies and provides an easily interpretable indicator of the structural safety. The deterministic code-based approach is commonly used for the in-practice assessment of existing bridge structures (e.g. BMVBS 2011) and therefore this approach is a straightforward method for strategy assessment with respect to structural safety

(Denton 2004: 8). In the scope of this thesis, the ratio $\eta$ will be labelled by the term *degree of fulfilment*.

$$\eta = \min_i \left( \eta_{Elem,i} \right) \qquad \text{with} \quad \eta_{Elem,i} = \frac{R_{d,Elem,i}}{S_{d,Elem,i}} \tag{2.6}$$

with

| | |
|---|---|
| $\eta$ | degree of fulfilment (overall structure) |
| $\eta_{Elem,i}$ | degree of fulfilment (i$^{th}$ element, e.g. 'girder in span 1') |
| $R_{d,Elem,i}$ | code/standard-based strength (e.g. bending) of i$^{th}$ element |
| $S_{d,Elem,i}$ | code/standard-based load-effect (e.g. bending moment) of i$^{th}$ element |

**Probabilistic Approach**

The alternative *probabilistic approach* calculates the structural reliability of the bridge with the help of stochastic methods. In general three methods for probabilistic safety assessment with different levels of complexity can be distinguished.

The most advanced and sophisticated method is the direct calculation of the reliability by simulation-based assessment of an FE-model of the overall structure. This is usually achieved by first generating a limit state surface from sampling points that result from FE calculations. For this limit state surface the structural reliability is calculated by simulation procedures, such as Monte-Carlo simulation or Latin hypercube sampling (e.g. Melchers 1999: 168–172; Dette et al. 2010). Such a sophisticated reliability assessment has been conducted, for example, by Strauss et al. for a major motorway viaduct in Italy (Strauss et al. 2008, Strauss and Bergmann 2008, Bergmeister and Santa 2004: 420–421).

A simplified method is the decomposition of the structural system into components and the calculation of the failure probability for each element. The overall reliability for the bridge is then calculated based on the logic arrangement of the individual components in serial and parallel order. The failure probability of the components is usually calculated by simulation-based (e.g. Monte Carlo) investigation of the components' limit state functions in the form of G=R-S. This method is, for example, applied for the assessment of maintenance strategies by Radojicic (2002) and also by Frangopol and Estes (1997).

Finally, the simplest method for estimating the structural reliability is to calculate it on the basis of assumptions regarding the distribution type and parameters of the components strength and their loading, for example their mean value and standard deviation. If both S and R are normally distributed, the reliability $\beta$ can be calculated as follows (Melchers 1999: 18):

$$\mu_G = \mu_R - \mu_S \quad \text{and} \quad \sigma_G^2 = \mu_R^2 + \mu_S^2 \tag{2.7}$$

$$\beta = \frac{\mu_G}{\sigma_G} \tag{2.8}$$

with

| | |
|---|---|
| $\mu_R$ and $\mu_S$ | mean value of the strength and the loading |
| $\sigma_R^2$ and $\sigma_S^2$ | variance of the strength and the loading |
| $\mu_G$ and $\sigma_G^2$ | mean value and variance of the limit state function |
| $\beta$ | reliability index |

Irrespective of the approach that is used for strategy assessment—deterministic or probabilistic—the structural safety (or reliability in case of the latter) must be repeatedly calculated during the reference period for strategy assessment. This is because deterioration mechanisms and a possible increase in traffic loads as well as the application of maintenance and repair measures lead to time-dependent changes of the bridge safety.

However, the time-dependent calculation of the structural reliability can be very work-intensive—especially in the case of a direct reliability calculation by means of simulation methods—and thus can make the assessment of strategies very tedious or even unreasonable. A significant improvement of the time necessary for the calculation can be achieved by applying the sophisticated approach *High Dimensional Model Representation* for structural reliability analysis (e.g. Chowdhury et al. 2009, Chowdhury and Rao 2010), which effectively provides a simple but at the same time highly accurate approximation of the structural limit-state function and thus reduces the time necessary for stochastic reliability analysis. However, this approach still necessitates a stochastic analysis at close time-steps for expressing the time-dependent changes that result for example from deterioration mechanisms and maintenance interventions.

As a solution to this, some authors approximate the time-dependent development of structural reliability by $\beta(t)$ curves in the form of $\beta=\beta_0-\alpha(t-t_0)$ (e.g. Petcherdchoo et al. 2008; Neves and Frangopol 2005). This is an analogous method to the above described simplifying calculation of an overall condition index (equation 2.1). These $\beta(t)$ curves are basically linear regression curves for the results of repeated reliability evaluation for the deteriorating parameters and are very simply expressed by a starting value ($\beta_0$) and linear deterioration rates ($\alpha$), as it is illustrated in Fig. 2.8. Also Denton (2004: 8) applies such a simplifying expression for a deterministic code-based expression of the structural safety in the form of a *C*(*t*) curve, where the C is an expression equivalent to the above described degree of fulfilment $\eta$.

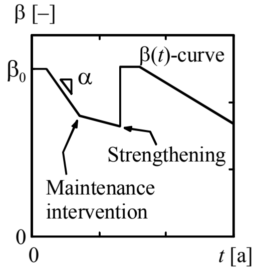



**Fig. 2.8:** Approximate representation of the time-dependent development of structural reliability by a multi-linear β($t$)-curve.

However, as it is the case with the above discussed assessment with respect to the bridge condition, the modelling of the maintenance effects on the bridge reliability (or safety) is quite opaque because in this case the maintenance effect must be described with respect to the development of the overall reliability index, even if the maintenance activity only affects just individual elements (e.g. Denton 2004: 47–50 or Neves and Frangopol 2005: 193). Consequently, this approach is also preferably used for investigations that refer to whole bridge stocks (Denton 2004) or to large groups of bridge elements, such as concrete crossheads (Neves and Frangopol 2005, Neves 2005), or bridge girders and deck slabs (Petcherdchoo et al. 2008). Just like the overall condition assessment approach (sub-section 2.7.2), this approach is not suitable for detailed investigations regarding maintenance strategies for individual bridges due to its very general character.

A possible solution for individual bridges, which provides estimates for the structural reliability as a function of multivariate set of time dependent variables—such as material properties, structural dimensions, and load—has been developed by Dette et al. (2010). This approach proposes the application of a meta-response surface—the so called *Reliability Response Surface* (RRS)—which effectively gives the reliability for a multivariate set of time dependent variables (see Fig. 2.9 (b)). This RRS is basically an expansion of the above described β($t$) curve concept to a larger number of variables, including material properties and structural dimensions. In this method, the development of the variables can be modified independently. As a result, the effects of maintenance measures on subsets of the variables can be accounted for in a consistent way, and thus the weak point of the β($t$) curve concept is overcome. A preliminary example application shows that the RRS can significantly accelerate the process for simulation of maintenance strategies. The example application also indicates that the results in terms of structural reliability show good agreement with results from direct structural-stochastic analysis (Dette et al. 2010).

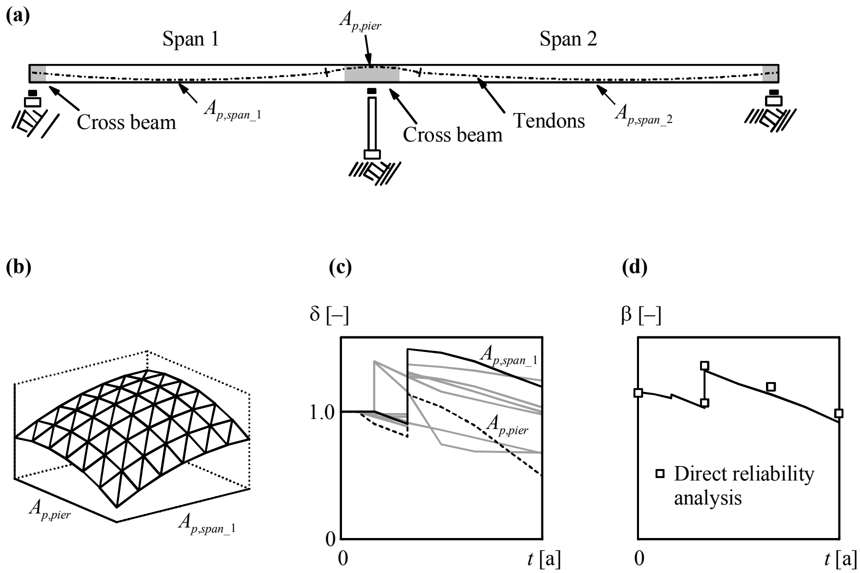



**Fig. 2.9:** Reliability Response Surface (RRS) approach: (a) Example bridge; (b) Example projection of a 24-dimensional RRS for this bridge, which provides β values as a function of 24 variables—ranging from truck loads to the girder shear reinforcement at the right hand support—in the two-dimensional plane of the amount of prestressing steel at the pier ($A_{p,pier}$) and at the low point of span 1 ($A_{p,span\ 1}$); (c) Time dependent development of selected variables; (d) structural reliability β($t$) as it is calculated with the help of the RRS (line) and direct reliability analysis (scatter points). [These figures are based on figures 4 and 6–8 in Dette et al. 2010]

### 2.7.4 Strategy Assessment with Respect to Economic Aspects

The economic impact of maintenance strategies is usually assessed with respect to two basic types of costs: *internal* and *external costs*. The first costs are linked to payment transactions and have to be carried directly by the owner of the bridge. Therefore, some authors label them as *agency costs* or *owner costs*. The term *owner costs* is used in the scope of this thesis in order to refer to this type of costs. As opposed to these internal costs, many authors use the term *external costs* to account for adverse impacts for the bridge users, the local residents, the society as a whole, or the environment in monetarised form. Most of these monetarised effects relate directly to the users of the bridge, therefore these costs are often referred to as *user costs*, a term which is also used in present thesis.

The *owner costs* are commonly used for the economic assessment of maintenance strategies (Patidar et al. 2007; Branco and de Brito 2004; Frangopol and Liu 2007; Frangopol and Estes 1997). They comprise all costs in connection with the design, construction, inspection, maintenance and decommissioning of the bridge. Because these costs occur at different points in time, they are usually taken into account in terms of their *net present value*. Some authors also use the economic indicator *annuity*, which expresses a fictional constant annual payment over the entire service life whose net present value is equal to the actual series of maintenance expenditures. In both cases, the owner costs can be determined by simple financial calculus following equations (2.9) and (2.10).

$$C_{0,owner} = C_{d+c} + \sum_j \left[ C_{maint,j} \left(1+i\right)^{-T_{app,j}} \right] + C_{decom} \left(1+i\right)^{-T_{SL}} \tag{2.9}$$

$$a = C_{0,owner} \times \frac{i\left(1+i\right)^{T_{SL}}}{\left(1+i\right)^{T_{SL}} - 1} \tag{2.10}$$

with

| | |
|---|---|
| $C_{0,owner}$ | net present value of total owner costs (time base: year of completion) |
| $C_{d+c}$ | costs for design and construction (time base: year of completion) |
| $T_{app,j}$ | time of application of measure $j$ (years after completion) |
| $C_{maint,j}$ | costs for maintenance measure $j$ at $T_{app}$ |
| $T_{SL}$ | service life (years after completion) |
| $i$ | discount rate |
| $C_{decom}$ | costs for decomissioning of the bridge |
| $a$ | annuity |

When the indirect *user costs* are concerned, the approaches of different authors do not show such a consistency as it is the case with the above described owner costs. While there is general agreement on the composition of the owner costs, the user costs are defined quite individually. Furthermore, the distinction between monetary external effects and non-monetary costs varies from author to author. For example Patidar et al. (2007) consider traffic accidents and delays in monetary terms as user costs whereas Lounis and Daigle (2010) assign these aspects to social performance and do not superimpose them with economic costs. However, the time loss due to reduced speed because of maintenance interventions is treated by Lounis and Daigle in monetarised and discounted form as user cost. Finally, Branco and de Brito (2004) treat all external effects such as architectural / cultural and historic aspects as well as traffic delay and accidents as external costs expressed in monetary terms.

There is general agreement to use the indicators *net present value* or *annuity* of the owner costs for economic strategy assessment. Both indicators condense all service life owner costs to in a single value and thus are perfectly suitable for strategy assessment over the whole service life. However, the strategy assessment with respect to user costs turns out to be a disputed topic. This is mainly because the external effects are often not connected with direct payment transactions and the necessary process of evaluating societal and environmental effects in monetary terms is rather controversial: What is the price of human life? What is the price of 1 m³ CO2 released into the atmosphere or of 1 hour leisure time? Furthermore, the superposition of internal and external costs causes difficulties due to the fact that the magnitudes can differ widely (Graubner et al. 2010; Fischer et al. 2010; Wallbank et al. 1999). Another debated aspect is the discounting of external costs by which future gains and losses are lower valuated: In contrast to owner costs, where discounting is an acknowledged procedure for intertemporal comparison, the lower valuation of future losses and gains leads to controversy when environmental aspects or human life and health is concerned (Peake and Smith 2009: 182–186, 201).

### 2.7.5 Strategy Assessment with Respect to Societal and Cultural Aspects

Bridge structures interact with their surroundings in various ways — for example in terms of effects on traffic, aesthetic impact, prestige and historic value. The functioning as a traffic route, the structural capacity and the actual condition are determinant for the way a bridge meets the mobility demands of the society. A reduced capacity due to maintenance works for instance directly affects the users. Also a deteriorated road condition on the bridge and debris from spalling concrete, which falls on underpassing roadways, can cause traffic accidents. Another important point is that bridges are often located directly in the urban space or are seen as prestigious landmarks. Therefore, their aesthetic impact and acceptance is another important performance aspect (Bonenberg 2010). This can considerably be affected by cracking and spalling of the concrete. Furthermore, historic bridges can constitute a part of the cultural heritage and hence, preservation can become an important aspect.

A review of selected papers on investigations of maintenance strategies shows that the following societal and cultural aspects are generally considered for strategy assessment:

- traffic related aspects (congestion, detour, traffic capacity, accidents)
- preservation of heritage and remaining service life
- aesthetic impact

However, it also appears that the relevant literature treats societal aspects very inconsistently when it comes to explicit strategy assessment. A common way to account for these aspects is to use an aggregate indicator that merges all societal aspects in monetarised form, which is commonly applied in the case of traffic related effects. Obviously, this method does not allow for a detailed assessment of individual societal aspects. Occasionally the societal aspects are also combined with monetarised environmental performance measures so that even no differentiation between these two aspect categories is achieved. Only exceptionally isolated societal aspects—in this case traffic flow and traffic accidents—are treated in terms of individual performance indicators. However, aspects regarding heritage preservation and aesthetic impact are not found to be explicitly considered for strategy assessment. Instead, these aspects are usually indirectly treated in terms of service life requirements and an overall condition index.

*Traffic aspects*, regarding traffic flow, detour, congestion and accidents, are often considered for the assessment of maintenance strategies. However, these aspects are usually treated in monetary terms, as it is shown above for the economic aspects. Moreover, many authors superimpose all the monetarised effects, including both owner and user costs and environmental effects, in order to form a single cost indicator for strategy assessment. In this case a differentiated strategy assessment of the societal impact is not possible.

Lounis and Daigle (2010) conduct a very detailed societal impact assessment of alternative bridge deck layouts, which also includes the effects of the layout-specific maintenance activities on traffic flow and the accident rate. This is done by a detailed calculation of lost user time because of reduced travel speed from maintenance activities. The cumulative lost time is explicitly used for strategy assessment. This is done in parallel to a common assessment of user costs including vehicle operating costs and the value of the driver's time. Traffic accidents are accounted for by calculating the increased number of accidents during maintenance works multiplied with average costs of 33,000 $ per accident. However, as these accident costs are treated separately from the economic indicator 'service life costs', a proper individual assessment of the societal aspect 'traffic accidents' is conducted by Lounis and Daigle.

A less detailed approach is applied by Pommerening et al. (2008: 39) and Haardt (2002: 15–20) who consider owner and user costs separately for strategy assessment. Both calculate the costs according to the *EWS Empfehlungen für Wirtschaftlichkeitsuntersuchungen bei Straßen* (suggestions for economic investigations of roads, EWS 1997), which provide detailed guidelines for the monetarised consideration of traffic flow effects—congestion and detour in terms of lost travel time and additional vehicle operating costs and fuel consumption—and all kinds of traffic accidents. Therefore, this approach only allows for a general assessment of the societal impact of alterative maintenance strategies where all societal effects are condensed to one single indicator: the user costs.

Finally, Denton (2004: 45–50) and Neves (2005: 55–58) do not provide a differentiate consideration of societal aspects. They both combine all costs—user and owner—for each maintenance activity into one single cost value and calculate on this basis an overall purely economic indicator for strategy assessment.

An explicit strategy assessment with respect to *heritage preservation* was not found in the reviewed literature. Naturally, conservation of historic or monumental bridges, which according to Tilly (2002: 1) includes special restoration and preservation activities, deals with more than just reaching a long service life. Yet, a long required service life is a common aspect of the maintenance management of historic or monumental bridges. The service life is occasionally used for strategy assessment but mostly as a requirement or an auxiliary variable in the overall assessment procedure rather than an explicit assessment indicator.

A direct strategy assessment with respect to the service life as an independent assessment-indicator is conducted by Furuta and Kameda (2006). They propose in their paper an algorithm for multi-objective maintenance planning, which considers service life as a specific indicator for assessment and evaluation in addition to life cycle costs and structural safety.

Pommerening et al. (2008: 65–67) implicitly consider the service life for strategy assessment by assessing maintenance strategies with respect to the financial indicator *annuity*, which includes the service life in the calculation according to equation 2.10.

However, most authors consider the service life as a requirement in the form of a minimum service life (e.g. Frangopol and Estes 1997), which must be fulfilled by the strategy but is not relevant for further strategy assessment and evaluation. Some of these authors (e.g. Neves and Frangopol 2005, Neves 2005 and Denton 2004) even include a *complete rebuild option*, in which case the bridge becomes a merely functional entity, which lives on when replaced by a new structure.

The *aesthetic bridge performance* is also not found to be explicitly assessed in the relevant literature on maintenance strategy assessment, even if many authors mention this aspect to be important (e.g. fib 2008a: 91, 101; fib 2002: 59). Lauridsen reports that aesthetic aspects, together with other non-monetary aspects, are taken into accounted for strategy selection within the Danish BMS DANBRO, however he also states that these factors are '*not included in the automatic* [strategy assessment and optimisation] *process*' (Lauridsen 1999: 65).

Only a general assessment of the condition, which naturally includes aesthetic aspects, is commonly conducted: While Neves and Frangopol (2005), Neves (2005) and Denton (2004) apply only one overall condition index, Pommerening et al. (2008: 15–28, 51–65) apply a more detailed consideration of individual element groups, for example the pavement, the soffit and the upper side of main girders as well as the transverse girders.

However, the condition index, as it is commonly defined, is not an explicit measure of the aesthetic performance. It is rather an aggregate measure of the overall deterioration level, which merely includes aesthetic aspects beneath many other aspects. Especially in the work of Pommerening et al. the condition index rather represents technical issues, such as the depth of chloride penetration into concrete and the further reinforcement corrosion progress or the general degradation of bridge equipment parts, as well as the corrosion of structural steel (Pommerening et al. 2008: 25–26, 31).

### 2.7.6 Strategy Assessment with Respect to Environmental Aspects

Environmental aspects have gained more and more attention over the last years, especially in connection with the sustainability debate. The relevant literature (e.g. Graubner et al. 2010; fib 2008b; DAfStb 2007) provides a comprehensive set of methods for assessing environmental impacts. In general, the overall environmental impact of maintenance strategies, which results from specific impacts, such as $CO_2$-emissions and waste production, can be assessed by two methods (see Fischer et al. 2010; fib 2008b: 25–30; DIN EN ISO 14040: 25–31; DIN EN ISO 14044: 33–43; Branco and de Brito 2004):

- impact categories
- eco-indicator method (special case: monetarisation)

The strategy assessment with respect to *impact categories* assesses the environmental impact individually with respect to typical distinctive environmental impact categories (DIN EN ISO 14044: 13). Typical impact categories are for example *global warming*, *resource use*, *waste production*, and *eutrophication* (e.g. fib 2008b; ISO/TR 14047). These aspects are often represented by performance indicators like *$CO_2$ equivalent emissions* (kg), *primary energy consumption* (J), *landfill* ($m^3$) and *$PO_4$ equivalent* (kg). In this case, the overall environmental impact is expressed by an *environmental impact profile*. These profiles describe, in analogy to the preference- or strategy performance profiles (see Definition 13 and Definition 18), the overall impact in terms of all individual values of the relevant environmental indicators.

This method is applied for strategy assessment by Lounis and Daigle (2010), who use two individual environmental impact categories for maintenance strategy assessment: $CO_2$-equivalent emissions and waste production due to replacement of asphalt overlay and concrete repair.

The *eco-indicator* method assesses the environmental impact in terms of a single score, the eco-indicator, which expresses the aggregate impact of all individual categories (fib 2008b: 25). This resulting indicator is usually calculated by normalisation and weighted superposition with respect to the individual categories (e.g. fib 2008b: 25, 29–30; DIN EN ISO 14044: 41–43). A special type of aggregating is to express the overall impact in terms of a monetary value (*monetarisation*). However, the expression of environmental impacts in terms of monetary value is quite controversial—the same way it is the case with societal and cultural issues. Especially appropriate and consistent monetary values for the different impacts have not yet been provided by the relevant literature. For example there is a considerable variation in the monetary assessment of $CO_2$-equivalent emissions, which ranges from 15 €/t to 150 €/t (Graubner et al. 2010). Moreover, handling the environmental aspects in monetary terms implies that they can be superposed with real monetary owner costs. However, such a superposition can be quite distortive because the magnitudes of both costs can differ widely (Graubner et al. 2010; Fischer et al. 2010; Wallbank et al. 1999). This effect is even stronger if future environmental costs are not discounted: In contrast to owner costs, where discounting is an acknowledged procedure for intertemporal comparison, the lower valuation of future losses and gains leads to controversy when environmental aspects or human life and health is concerned (Peake and Smith 2009: 182–186, 201).

This method is applied for strategy assessment by Denton (2004), Branco and de Brito (2004), and Pommerening et al. (2008) who all evaluate the environmental impact, especially from $CO_2$ emissions, in monetary terms.

## 2.8 Evaluation of Maintenance Strategies

### 2.8.1 Overview

The process of *strategy evaluation* investigates how well a strategy is suitable for reaching the maintenance objectives. This is clearly a multi-objective evaluation process because usually many different—and often competing—maintenance objectives must be considered. More specifically, this process can be described as the valuation of the strategy performance profiles (see the above Definition 18) with respect to the individual objective profile (see the above Definition 13). Accordingly the following definition for strategy evaluation will be used in the scope of this thesis:

> **Definition 20**: The term **strategy evaluation** describes the valuation of the strategy performance profile with respect to the maintenance objective profile of the bridge. Such an evaluation procedure is usually of multi-objective type because of the many different performance aspects that must be taken into account.

From the relevant literature, three common methods for multi-objective strategy evaluation can be found for the bridge specific context:

- *isolated consideration* of the multiple objectives
- *monetarisation*, possibly together with the definition of benefit-cost ratio indicators
- *non-monetary transformation* to a unidimensional problem

In the following, these methods are discussed with respect to their advantages and shortcomings, especially in the field of bridge maintenance. On the basis of these findings, an alternative method—*Qualitative Preference/Performance-Profile Comparison*—is proposed, which overcomes the shortcomings that are identified for the three commonly applied methods.

### 2.8.2 Isolated Consideration of Multiple Objectives

Strictly speaking, the isolated consideration of multiple objectives does not qualify as true multi-objective evaluation. It exceptionally works in cases of strict domination, when a strategy excels all alternative strategies with respect to each performance aspect. It also exceptionally works when all but one of the objectives are considered only in such a way that they must satisfies certain minimum requirements and only one isolated aspect is applied for evaluation purposes.

This methodology is applied for example by Denton (2004), Neves (2005) and Neves and Frangopol (2005), who evaluate maintenance strategies in terms of life cycle costs, structural safety and condition. They evaluate the strategies in an isolated way in terms of three performance aspects—safety, condition, costs—but do not evaluate the whole picture. Crucially, the pareto-solution seeking optimisation methods applied by Neves and Frangopol (2005) must not be confused with true multi-objective strategy evaluation. This is because as a result of strategy investigation they provide a set of indicator-value combinations, with each one being a pareto-solution in such a way that for each combination there is no way of altering the maintenance strategy so that one of the indicator-values is improved without at least one of the other values being necessarily worsened. Consequently, Neves and Frangopol provide the important first step of finding the—theoretically infinite—set of optimal strategies that provide the best results for one indicator without compromising the results of the other indicators. However, they do not provide a methodology for how to identify from this set the solution that fits best to the specific bridge under consideration of the preference profile.

The same applies to Lounis and Daigle (2010), who evaluate bridge deck design alternatives in a service-life perspective: Although they explicitly consider a broad range of economic, societal and environmental aspects, they only compare individual performances—which, however, is perfectly sufficient in their example evaluation because one design alternative clearly dominates the other alternative with respect to all performance aspects.

### 2.8.3 Monetarisation and Benefit-Cost Indicators

A commonly applied method for multi-objective evaluation is to transfer the underlying multidimensional problem to a unidimensional problem by introducing a function that transforms the individual performance indicators into a single resulting performance indicator. Then, this resulting indicator represents the aggregate performance and serves as a measure for comparing alternative strategies.

The most common procedure is to express all performance aspects in monetary terms and then to evaluate the maintenance strategy with respect to the sum of all costs (e.g. Fischer et al. 2010: 4). The monetarisation of the external effects is usually achieved by assigning monetary values to the adverse societal, cultural and environmental effects. Typical adverse effects are, for example, the lost time due to traffic congestion and detour, traffic accidents, climate change, and a loss of architectural value.

$$C_0 = C_{0,owner} + C_{0,external} \tag{2.11}$$

with

$C_{0,owner}$ net present value of total owner costs

$C_{0,external}$ net present value of total societal/cultural/environmental costs

(time base for all costs: year of completion)

However, this simple additive superposition of real costs with the imaginary costs that arise from monetarised societal, cultural and environmental external effects is quite controversially discussed (see sub-section 2.7.4 and Peake and Smith 2009: 182–186, 201). Particularly, it must be kept in mind that the theoretical external costs can exceed the owner costs by far so that the external effects may play an all-dominant role in case of simple summation of costs (Fischer et al. 2010: 4). Moreover, it must be taken into account that different parties are bearing the costs (e.g. Graubner et al. 2010: 337; Fischer et al. 2010: 4). Consequently, the absolute cost value must always be interpreted with respect to a reference value, for example the owner costs with reference to the overall maintenance budget and the external user costs with respect to the regional gross domestic product.

A solution to this problem is to express all relevant aspects in monetary terms and evaluate the alternative strategies with respect to an indicator describing the ratio of *life cycle owner costs* to *life cycle user costs*. By this method the problem of different dimensions is solved together with the separation of the affected parties (owner and user/society/environment). This method is used, for example, by Pommerening et al. (2008: 41–48), Radojicic (2002: 115–125), Haardt (2002), and Branco and de Brito (2004). By this method all non-monetary aspects—societal, cultural and environmental—are assessed in monetary terms and then put into relation to the (monetary) owner costs, which include the expenses for design, construction, maintenance and replacement or decommissioning. Typically the indicator for strategy evaluation is chosen in the form of a '*benefit-cost-ratio*', where the cost are defined as the owner costs and the benefit is the reduction of the user costs with compared to a reference value. Pommerening et al. (2008) use the strategy with the lowest owner costs for defining the reference costs, while Radojicic (2002) defines the reference value as the costs that potentially arise from structural failure and functional restrictions.

$$BCR = \frac{B_{0,user}}{C_{0,owner}} = \frac{C_{0,user,reference} - C_{0,user}}{C_{0,owner}} \tag{2.12}$$

with

| | |
|---|---|
| $BCR$ | benefit-cost-ratio |
| $B_{0,user}$ | user benefit (in monetary terms) |
| $C_{0,owner}$ | net present value of total owner costs (for the *evaluated* strategy) |
| $C_{0,user}$ | net present value of total user costs (for the *evaluated* strategy) |
| $C_{0,user,reference}$ | net present value of total user costs (for *reference* strategy) |
| | (time base for all costs: year of completion) |

However, the fundamental weakness of this practical evaluation method is that all non-monetary aspects are put together in one single value. Again, this is subject to debate because of the different cost-dimensions of societal, cultural and environmental aspects. To account for this, individual benefit-cost-ratios for each aspect must be defined—but in this case the problem stays multi-dimensional.

A solution to this is described in DAfStb (2007: 36), which uses the ratio of the *aggregate non-monetary aspects* and the monetary owner costs and in terms of a non-monetary benefit-cost analysis.

$$BCR_{nonmonetary} = \frac{B_{0,nonmonetary}}{C_{0,owner}} = \frac{I_{nonmonetary,refernce} - I_{nonmonetary}}{C_{0,owner}} \tag{2.13}$$

with

| | |
|---|---|
| $BCR_{nonmonetary}$ | benefit-cost-ratio with respect to non-monetary aspects |
| $B_{0,nonmonetary}$ | benefit with respect to non-monetary aspects |
| $C_{0,owner}$ | net present value of total owner costs (*evaluated* strategy) |
| $I_{nonmonetary,refrence}$ | aggregate adverse impact w.r.t. non-monetary aspects (*reference* strategy) |
| $I_{nonmonetary}$ | aggregate adverse impact w.r.t. non-monetary aspects (*evaluated* strategy) |

However, this is fundamentally different compared to the above described method because it needs the definition of a non-monetary aggregation function. Such a non-monetary function is discussed in the following sub-section.

### 2.8.4 Non-Monetary Transformation Functions

The application of non-monetary transformation functions for the evaluation of the life cycle performance is discussed primarily in connection with the sustainability debate (e.g. fib 2008b: 23–30; DGNB 2009). In this field sophisticated transformation functions are needed because sustainability evaluation encompasses quite different—sometimes even competing—issues from the fields of technology, economy, society, and environment.

The classic procedure is to transform each of the individual performance results to a uniform rating scale (e.g. 0 to 1) and calculate on that basis a resulting indicator by weighted superposition of the transformed performance indicators. The calculation by weighted superposition can be achieved, for example, in the form of the following *additive value function* (e.g. Keeney and Raiffa 1993: 117):

$$u(\boldsymbol{x}) = u(x_1, x_2, ..., x_n) = \sum_{1}^{n} w_i u_i(x_i) \tag{2.14}$$

with

| | |
|---|---|
| $u(x_1, x_2, ..., x_n)$ | value function representing the resulting overall performance |
| $x_i$ | indicator representing the i$^{\text{th}}$ performance aspect |
| $w_i$ | weighting factor representing the importance of the i$^{\text{th}}$ performance aspect with $\sum_{\mathrm{i}} w_i = 1$ |
| $u_i(x_i)$ | normalising value function representing the performance w.r.t. aspect $x_i$ |

Here, the *value function* $u_i(x_i)$ expresses the normalised valuation of the performance with respect to the individual aspect $x_i$. A practical transformation is to assign a value of ‘0’ to the lowest observed performance and a value of ‘1’ to the highest performance, which is observed for the decision alternatives (e.g. Keeney and Raiffa 1993: 140; 369–376). Such value functions can take many different shapes. Typical basic types of shapes are: concave, convex and linear (Fig. 2.10).

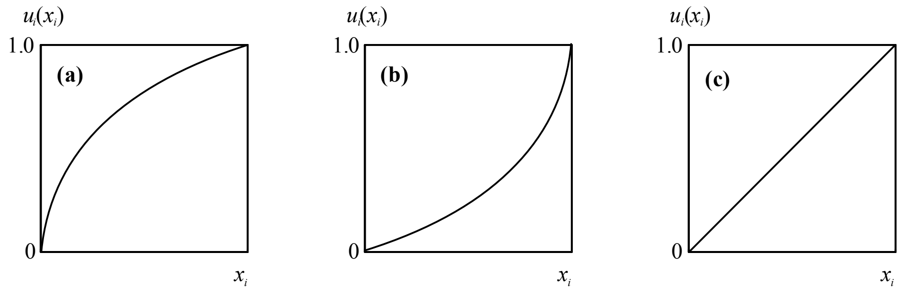



**Fig. 2.10:** Example curves of example value functions $u_i(x)$: (a) concave, (b) convex, (c) linear.

The *concave* value function expresses *risk-aversion*, where the decision taker prefers decisions that lead to certain results in favour of decisions that may lead to better results of $u_i(x)$ but also include the risk of lower $u_i(x)$. The convex value function, on the other hand, expresses *risk-proneness* where the decisions that lead to possible better results are preferred to a decision leading to certain, but intermediate, results. Finally, the *linear* value function expresses risk-neutral behaviour.

The *weighting factors* $w_i$ account for the relative importance of the individual performance aspects (e.g. Keeney and von Winterfeldt 2007: 245). For example, the weights may describe how much more important an extension of the service life is compared to savings in maintenance cost.

Procedures for the development of the value function in terms of the parameters $w_i$ and $u_i$ are described in detail by Keeney and Raiffa (1993). They encompass complex interviews with the decision makers in order to find out their value judgements and thus can be very time intensive (Keeney and Winterfeldt 2007: 246). However, the responsible decision makers sometimes are not willing to participate in such a time intensive detailed interviews. Therefore, often the value function must be developed on the basis of a few key value judgements obtained from simple and short interviews with the decision makers (Keeney and Winterfeldt 2007: 246–250).

As a solution to this, predefined valuation systems can be applied. For example the *DGNB Deutsche Gesellschaft für Nachhaltiges Bauen* (German Sustainable Building Council) provides a sophisticated rating system for office and administrative buildings (DGNB 2009). This system applies a weighted rating on two subsequent stages.

On the first stage several indicators are condensed by weighted additive superposition to a resulting rating for each of five individual performance aspect categories: technical quality, economic quality, socio-cultural quality, environmental quality, and quality of the planning and construction processes. For example, the rating of the environmental quality results from altogether 15 individual indicators, ranging from *global warming potential* to *land demand*, while the economic quality is calculated from the two indicators *building-related life cycle costs* and *suitability for third party use* (DGNB 2011: 27). The weighting factors for the superposition within the aspect categories are explicitly defined within the rating system, for example 0.15 in the case of the *global warming potential* and 0.10 for *land demand* (DGNB 2011: 27). The individual aspects, like *global warming potential* or *land demand* are normalised by rating them on a 1 to 10 scale. However, the individual valuation functions, as depicted in Fig. 2.10, for evaluating the observed performance are only exceptionally described by the DGNB—in general, the definition of the valuation function is left to personal and informal judgement. In exceptional cases, the valuation function is usually defined as being a linear transformation (compare Fig. 2.10 c)) which calculates a value $u_i(x_i / x_i^*)$ from the ratio of the performance $x_i$ to a reference value $x_i^*$. An example for this are the yearly $CO_2$-emissions which are related to a reference value (DGNB 2009: 14).

On the second stage, the ratings of the five performance aspects are condensed—again by weighted additive superposition—to an overall rating. Here the weighting factors are explicitly defined as being equally 0.225 except for process quality, which is weighted by 0.10.

Consequently, the overall rating process as it is proposed by the *Deutsche Gesellschaft für Nachhaltiges Bauen* (German Sustainable Building Council) can also be described by equation 2.14 because the above described two stages can be combined by multiplying the individual aspect weightings with the category weightings—for example, the overall weighting of the *global warming potential* can be calculated to 0.03375 (=0.225×0.15).

Graubner et al. (2010) discuss in their paper *Nachhaltigkeitsbewertung für die Verkehrsinfrastruktur* an approach to expand the above described DGNB- procedure, which is currently specific for office and administrative buildings, to infrastructure structures like bridges and tunnels. They argue that, due to its transparent and adaptable layout, the procedure can easily be adjusted to infrastructure evaluation. Of course it will be necessary to introduce specific new performance aspects, for example external costs due to traffic congestion, while other building-specific performance aspects, such as thermal comfort, can be discarded (Graubner et al. 2010: 333). Also the valuation functions and weighting factors should be adjusted to the special situations of infrastructure structures and should differentiate between structures, such as bridges, tunnels, and roads (Graubner et al. 2010: 333–334). However, they assume the given weighting factors and the described value functions to be sufficient for an application to infrastructure structures in a preliminary phase.

### 2.8.5 Qualitative Preference/Performance-Profile Comparison

The above described performance aggregation by means of monetary or nonmonetary transformation is contested, especially if the aggregation refers to significantly different issues. The systems scientist Hartmut Bossel, for example, emphasises that a unidimensional evaluation is generally inadmissible and can reasonably be applied only in special cases when all indicators are naturally measured in convertible units—this is, for example, the case when all indicators are naturally expressed in monetary value or in terms of energy consumption (Bossel 2004: 235). Overall indicators that are aggregated across inconvertible units by (non-) monetary value functions, must only be applied for general performance evaluation if it is guaranteed that each individual indicator satisfies certain minimum requirements (Bossel 2004: 245, 268–269). But even in this case, he emphasises, an overall indicator can be distortive because significant different combinations of individual aspect-performances can yield similar overall performance values (Bossel 2004: 266). This is a logical consequence of a weighted additive value function, which allows for a trade-off between the individual performances. For example it can be the case that a good overall performance merely results from an extremely good performance with respect to a single aspect while the other performances are indeed very poor. In such a case, the rational choice would be a slightly less overall performance in connection with a more balanced performance profile—and not the above described one-sidedly extremely well performing strategy.

As a solution to this, an alternative method is proposed in the scope of the present thesis. This method consists in comparing the strategy performance profiles to the bridge specific preference profile in a transparent way. This is done graphically with the help of star-plot diagrams, which depict the performance levels with respect to the different aspects and thus easily highlight specific strength and weaknesses. In addition, an overall performance indicator, which results from weighted superposition, is used for a plausibility check of the profile-shape analysis. In detail, the method involves the following steps:

- generation of strategy performance profiles by transforming the individual assessment results to a uniform rating scale with the help of a normalising value function $u_i(x_i)$
- generation of the preference profile in the same normalised rating range
- additional calculation of an aggregate performance indicator $u(\boldsymbol{x})$ by weighted superposition of the initially calculated $u_i(x_i)$-values—the weighting factors $w_i$ for the performance aspects $x_i$ are derived from the relative aspect valuations within the preference profile
- depiction of the profiles in the form of star plot diagrams
- qualitative preference/performance-profile comparison with the help of the star plot diagrams and plausibility check with the help of the aggregate performance indicator $u(\boldsymbol{x})$

The following example demonstrates the application of the newly proposed profile comparison method for the evaluation of three alternative maintenance strategies with respect to the four preference categories *society*, *environment, function*, and *economy*.

In this example, the three maintenance strategies are assessed over a defined time horizon with respect to four performance indicators: traffic restrictions and reduced speed in the form of cumulated hours of delay, $CO_2$-equivalent emissions, average condition Index ($CI_{av}$), and the net present value of owner costs ($C_{0,owner}$). The hypothetical results of the assessment are listed in the following Table 2.4.

**Table 2.4:** Example assessment results

| | Society | Environment | Function | Economy |
|---|---|---|---|---|
| **Strategy** | Delay [h] | $CO_2$ [to/m²] | $CI_{av}$ [-] [a] | $C_{0,owner}$ [€] |
| $S_1$ | 750,000 | 150 | 1.75 | 10,000,000 |
| $S_2$ | 1,000,000 | 210 | 2.35 | 7,500,000 |
| $S_3$ | 1,250,000 | 250 | 2.50 | 8,000,000 |

[a] 1.0 (excellent) to 4.0 (very poor)

The first step includes the application of a non-monetary transformation function. In this example this is achieved by using linear value functions $u_i(x_i)$, which assign a value of '1' to the lowest observed performance and a value of '5' to the highest performance (Table 2.5).

**Table 2.5:** Example values $u_i(x_i)$

| Strategy | Delay | $CO_2$ / m² | $CI_{av}$ | $C_{0,owner}$ |
|---|---|---|---|---|
| $S_1$ | 5.0 | 5.0 | 5.0 | 1.0 |
| $S_2$ | 3.0 | 2.6 | 2.6 | 5.0 |
| $S_3$ | 1.0 | 1.0 | 1.0 | 4.2 |

As a next step, the bridge preference profile is defined in the range from a lowest preference value of '1' to the highest preference value of '5'. In this example, a remote minor traffic link is considered, where cost efficiency is very important, while traffic delay effects do not play an important role (Table 2.6).

**Table 2.6:** Example preference profile

| Society | Environment | Function | Economy | Σ |
|---|---|---|---|---|
| 2 | 3 | 3 | 4.5 | 2+3+3+4.5 = 12.5 |

The weighting factors $w_i$ for calculating the aggregate performance $u(\boldsymbol{x})$ are derived from the preference profile (Table 2.6) by relating the aspect rating to the sum of all ratings. For example, the weighting factor for the economic performance is calculated to 0.36, which results from a rating of 4.5 divided by 12.5, which is the sum of the individual aspects' ratings. The aggregate performance $u(\boldsymbol{x})$ is calculated according to equation 2.14.

**Table 2.7:** Resulting performance $u(x_1\ x_2\ x_3\ x_4)$

| Strategy | Delay | $CO_2$ / m² | $CI_{av}$ | $C_{0,owner}$ | $u(x_1\ x_2\ x_3\ x_4)$ |
|---|---|---|---|---|---|
| $w_i$ | 0.16 | 0.24 | 0.24 | 0.36 | |
| $S_1$ | 5×0.16 | 5×0.24 | 5×0.24 | 1×0.36 | 3.6 |
| $S_2$ | 3×0.16 | 2.6×0.24 | 2.6×0.24 | 5×0.36 | 3.5 |
| $S_3$ | 1×0.16 | 1×0.24 | 1×0.24 | 4.2×0. 36 | 2.1 |

The next step, which includes the graphic representation of the strategy performance profiles and the preference profile, is depicted in the following Fig. 2.11.

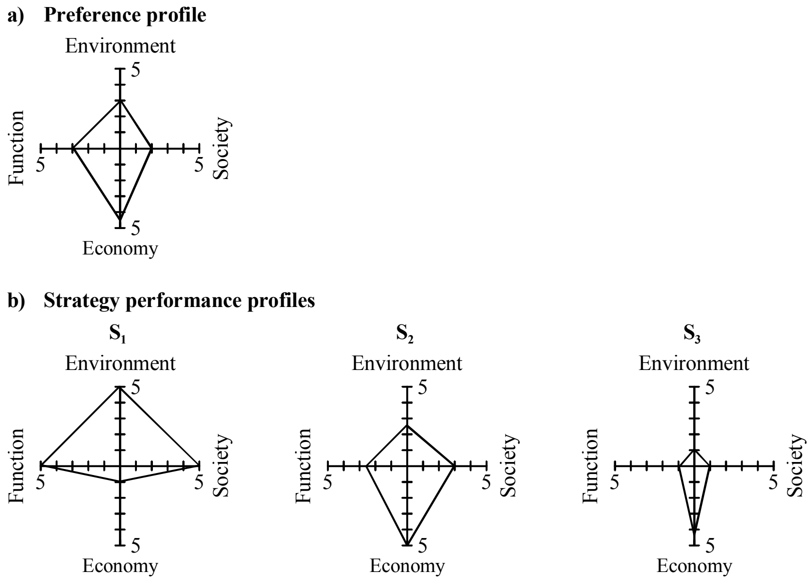



**Fig. 2.11:** Example profiles: a) preference profile; b) strategy performance profiles.

The qualitative profile analysis shows that strategy $S_2$ should be preferred because its performance profile is quite similarly shaped as the preference profile. Thus, the strategy shows specific strengths where they are needed and weaknesses where they do not matter. The strategy $S_3$ also shows a similar profile. However, it is dominated by $S_2$ with respect to all performance aspects. In contrast, the performance profile of strategy $S_1$ obviously counteracts the objective profile.

This example also illustrates the misleading trade-off effect between performance aspects, which can be successfully avoided by the newly proposed method of qualitative preference/performance-profile comparison: As can be seen from Table 2.7, the strategy $S_1$ seemingly excels the other two strategies because of its highest $u(\boldsymbol{x})$-value of 3.6. However, the graphic profile analysis in Fig. 2.11 reveals that on the contrary the strategy $S_2$ is the best choice and strategy $S_1$ is indeed rather inappropriate.

## 2.9 Conclusion

The present chapter provides an overview on the general bridge management process by defining and exemplifying the common terminology and applied concepts. Furthermore, the present chapter summarises and discusses the actual state of the art of bridge management and maintenance planning, as it is described in the relevant literature. It argues that bridges interact with the interests of many different parties and therefore maintenance planning must consider various aspects, which can be categorised into functional, economic, societal/cultural, and environmental issues. However, a review of the relevant literature on bridge maintenance planning reveals shortcomings with respect to a consideration of the whole set of the above mentioned performance aspects. Specifically, it occurs that practical approaches for the requirements and objectives development phase still are missing and also performance indicators for societal and other nonmonetary aspects and practical approaches for strategy evaluation should be developed. In case of the latter, the qualitative method of performance profile comparison, which is introduced in the present chapter as a general method for strategy evaluation, is a practical approach for strategy evaluation.

# 3 Survey of Bridge Management Practice

## 3.1 Introduction

This chapter describes a survey study of 24 bridges in Hamburg (Germany) and in Chennai (India). The objective of the survey is to create an empiric basis for the development of an approach to maintenance planning that accounts for multiple performance aspects and the respective bridge specific preferences. A focal aspect in this connection is to find indication for the theoretic assumption that the performance-preferences depend on the special situation of a bridge. As a second aspect, the survey investigates on which basis bridge managers decide for their maintenance application plans and if bridge specific maintenance strategies are practically implemented. Finally, the investigation provides insight on the methods for assessing and evaluating maintenance strategies, which are applied in engineering practice, and highlights potential fields for improvement.

## 3.2 Scope and Instruments of the Bridge Survey

### 3.2.1 Overview

The 24 bridges in Hamburg (Germany) and in Chennai (India) of this survey were selected in cooperation with bridge managers in both cities with the aim of representing characteristic types of bridges, such as landmarks, heritage bridges or mainly functional structures. The survey in Hamburg was conducted with the city's two major bridge authorities. One of these is responsible for the bridges of the Port of Hamburg while the other is responsible for the bridges belonging to the road and highway network in the city. In Chennai, one of the two cooperating authorities is responsible for the urban roads and bridges, while the other—the Highways Department— is responsible for the bridges along the important Anna Salai road.

The survey relies on two instruments. The first one is a multiple-choice questionnaire, which gathers information for identifying bridge specific performance preferences of stakeholders and bridge managers. In addition, it also identifies characteristic patterns of bridge specific maintenance strategies. Secondly, personal interviews with the bridge managers are used as a complementary instrument for gathering in-depth information on the topics of the questionnaires.

### 3.2.2 Hamburg Bridges

In Hamburg altogether fourteen bridges were investigated. Ten bridges are maintained by the *Landesbetrieb Straßen Brücken und Gewässer* (LSBG), which is responsible for all bridges belonging to the urban road and highway network. The remaining four bridges are managed by the *Hamburg Port Authority* (HPA), which manages the infrastructure of the Port of Hamburg. In detail, the following fourteen bridges were investigated (Fig. 3.1 and Fig. 3.2):

- Brücke K6 in Moorfleet (LSBG)
- Köhlbrandbrücke (HPA)
- Freihafen Elbbrücke (HPA)
- Zollenbrücke (LSBG)
- Lombardsbrücke (LSBG)
- Kennedybrücke (LSBG)
- Robert-Schumann Brücke(LSBG)
- Hoheluftbrücke (LSBG)
- Brücke Bahrenfelder Chaussee (LSBG)
- Hochstraße Elbmarsch (LSBG)
- 1. Peuter Brücke (HPA)
- Brücke Wördemanns Weg (LSBG)
- Brennerhofbrücke (LSBG)
- Brandenburger Brücke (HPA)

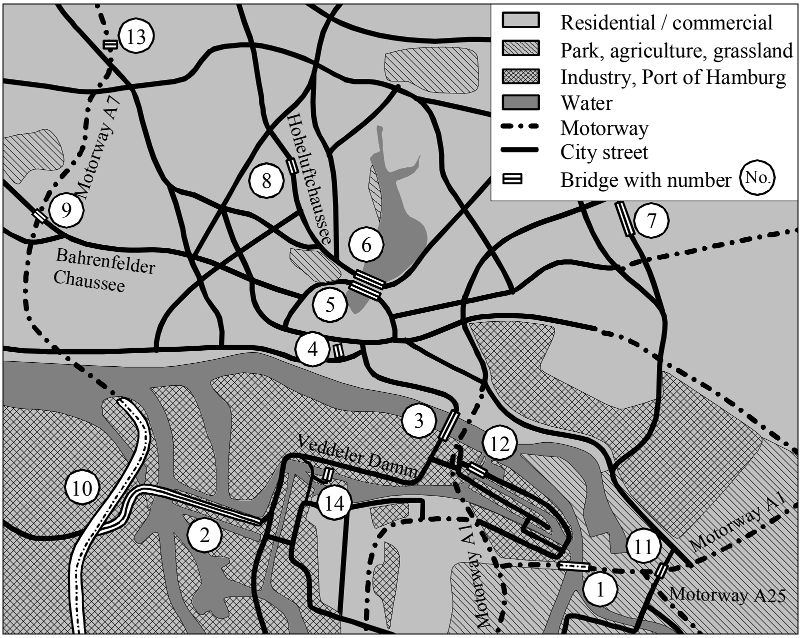



**Fig. 3.1:** Locations of the selected bridges in Hamburg: (1) Brücke K6 in Moorfleet, (2) Köhlbrandbrücke, (3) Freihafen Elbbrücke, (4) Zollenbrücke, (5) Lombardsbrücke, (6) Kennedybrücke, (7) Robert-Schumann Brücke, (8) Hoheluftbrücke, (9) Brücke Bahrenfelder Chaussee, (10) Hochstraße Elbmarsch, (11) Brennerhofbrücke, (12) 1. Peuter Brücke, (13) Brücke Wördemanns Weg, (14) Brandenburger Brücke

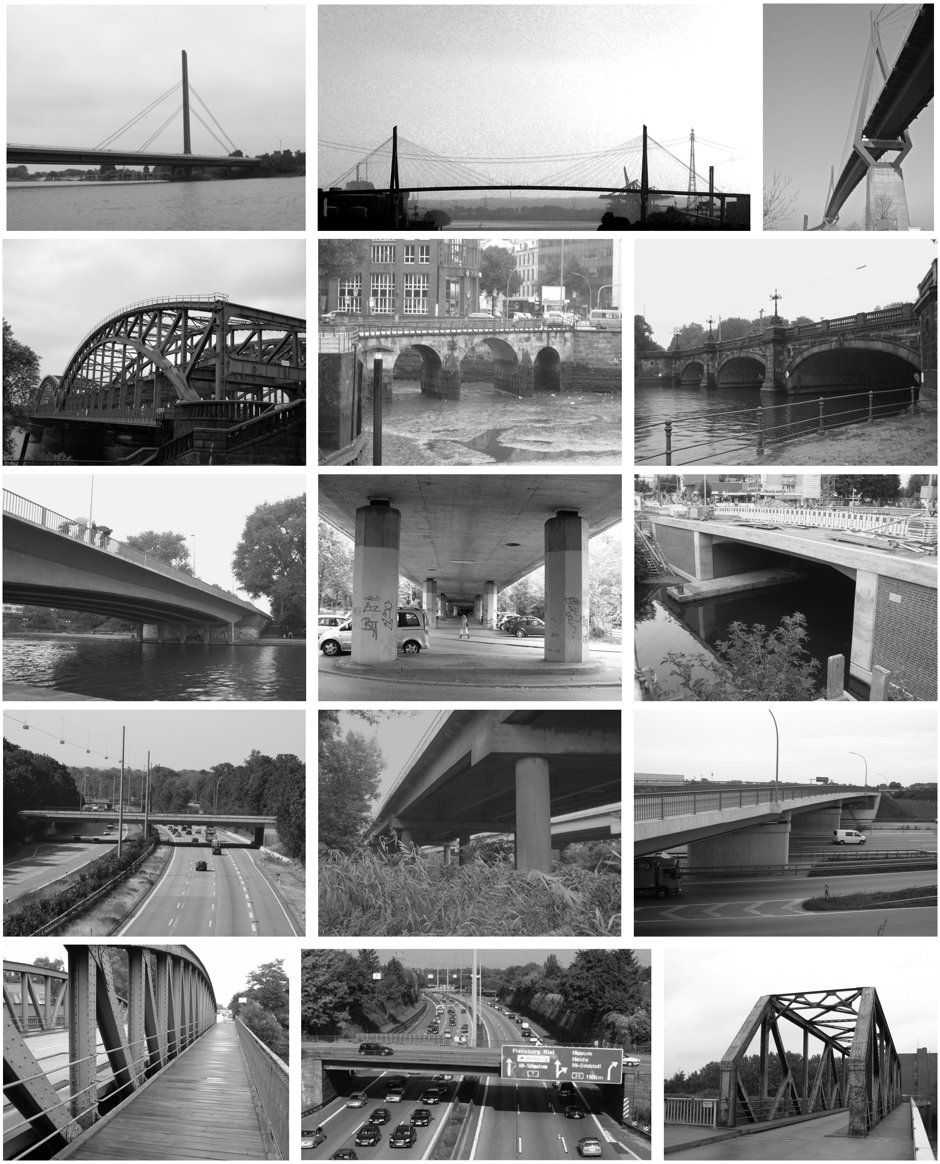


**Fig. 3.2:** Hamburg Bridges (from left to right): *First row*: Brücke K6 in Moorfleet, Köhlbrandbrücke (from north), Köhlbrandbrücke (from east with its characteristic pylons). *Second row*: Freihafen Elbbrücke, Zollenbrücke, Lombardsbrücke. *Third row*: Kennedybrücke, Robert-Schumann Brücke, Hoheluftbrücke. *Fourth row*: Brücke Bahrenfelder Chaussee, Hochstraße Elbmarsch, Brennerhofbrücke. *Fifth row*: 1. Peuter Brücke, Brücke Wördemanns Weg, Brandenburger Brücke (Photos: Grischa Dette, except for the third picture in the first row (Viktor Sigrist) and the central picture in the bottom row (LSBG))

The *Brücke K6 in Moorfleet* was built in 1963. It transfers the A1 motorway with 2×3 lanes across the Elbe River. The stay-cable bridge with a main span of 172 m and an overall length of 411m was designed by the structural engineers Heinz Aschenberg and Gerhard Freudenberg in cooperation with the architect Egon Jux, who was also involved in the design of the Köhlbrandbrücke (Hamburg 2002: 35, 45; Kurrer et al. 2010). Also the structural engineer Hellmut Homberg contributed to this formidable bridge both as a consultant and as a checking engineer (Kurrer et al. 2010). With just four stay cables per pylon it is a representative structure of the first generation of modern stay cable bridges, which emerged with Franz Dischinger's Strömsund Bridge in Sweden (1955) and Fritz Leohnhardt's Theodor-Heuss Brücke in Düsseldorf (1957) and lasted until 1967, when Hellmut Homberg introduced the innovative multi stay system with the erection of the Friedrich-Ebert-Brücke in Bonn (Kurrer et al. 2009 and 2010). The Brücke K6 in Moorfleet is an elegant landmark structure, which fits wonderfully in the green shoreland of the Elbe River in the south eastern part of greater Hamburg.

The *Köhlbrandbrücke* (1974) with its characteristic pylons is a well-known Hamburg landmark. It carries the main east-west traffic route with 2×2 lanes through the port of Hamburg and also connects the two motorways A7 and A1 (Gaffron et al. 2008). The main span of the stay-cable bridge is 325 m. Together with the prestressed-concrete ramps it is the second longest motorway-bridge in Germany with an overall length of about 3600 m. The main steel stay-cable bridge was designed by the structural engineer Paul Boué in cooperation with the architect Egon Jux—the same one who also took part in the Brücke K6 in Moorfleet, which comes apparent not least from the obliquely cut pylon heads of both bridges (see Fig. 3.2, first row). The design of the ramps was conducted under Hans Wittfoht, the technical director of the contractor Polensky & Zöllner. The designers of the stay-cable main bridge were given the European Steel Design Award for this outstanding structure. (Hamburg 2002: 45; Wittfoht et al. 1975; Wittfoht 1984; Boué and Höhne 1975a/b).

Another Hamburg landmark is the *Freihafen Elbbrücke*, which was built from 1916 to 1926 as a steel-arch bridge with an overall length of 471 m. It was originally designed in order to carry an elevated metro line on massive steel-frames above the road, which can be seen in Fig. 3.2—however, this metro line was never installed (Bardua 2009). The bridge connects the Port of Hamburg to the city centre in the form of a three-lane road and one railway track. Since its time of construction, the impressive bridge is located in the exclusively industrial surroundings of the Port of Hamburg. However, this will change from the year 2014 due to the development of the Baakenhafen area in connection with the development of the new HafenCity (HafenCity 2006). The future Chicago Square, the Elbbrückenzentrum, and the Quartier Baakenhafen will be characterised by high-class residential and commercial buildings as well as promenades along the Elbe River.

The *Zollenbrücke* is the oldest existing bridge in Hamburg and hence is listed in the city's heritage list (Hamburg 2010a). The arch bridge was built in 1633 from sandstone blocks with three arches and an overall length of 25 m. It was closed to motor traffic in 1953 and is restricted to pedestrian use since. (Benecke et al. 1988: 209–210; Lange 2008: 16).

The monumental *Lombardsbrücke,* which is also listed in the heritage list of Hamburg (Hamburg 2010a), was built in 1868 as a replacement for the then existing timber bridge with the aim to transfer the newly built railway line connecting Hamburg and Altona across the Alster Lake. Today, with an overall width of 48 m, it carries four railway tracks and a road with 2+3 lanes. The bridge was designed by the Hamburg building inspector Johann Hermann Maack in the form of a masonry arch bridge, which is cladded with blocks of granite. The three arches have an opening of 17 m each, resulting in on overall length of 69 m. In order to carry altogether 4 railway tracks, the bridge had to be broadened from 32 m to 48 m in 1901/02 by adding parallel arches (Lange 2008: 73; Benecke et al. 1988: 216–218; Stephan 1987).

The *Kennedybrücke* was built in 1953 based on the design by the engineering departments of the Hamburg building authority and the Dyckerhoff & Widmann construction company in cooperation with the architect Bernhard Hermkes (Krieger 1996: 116). The construction of the bridge under its former name *Neue Lombardsbrücke*—in 1966 the name was changed to Kennedybrücke in order to honour the assassinated US president—was necessary in order to relieve the heavily stressed Lombardsbrücke by four additional lanes for motor traffic. The prestressed concrete frame-bridge has a main span of 57.90 m; the total length and width are 87.40 m and 28 m respectively (Lange 2008: 73). The bridge is located parallel to the Lombardsbrücke at a distance of just 50 m so that the overall impression of the bridge can be obtained only from the side of the Außenalster, which is the vast lake in the centre of Hamburg. The bridge was of major interest at the time of construction in terms of functional and aesthetic aspects (Krieger 1996: 116). The soffit, which shows a curved transition from the closed box cross-section at the frame columns to the open multi-T-beam layout in the span, cites the well known kidney shaped tables of the late 1950s (Weber and Sigrist 2009). The bridge fits harmonically into its surroundings and is listed worthy of preservation in the heritage list of Hamburg (Hamburg 2010a).

The *Robert-Schumann-Brücke*, built in 1966, is a functional prestressed concrete road bridge with a box-girder cross-section. It transfers an arterial outer ring road on four lanes across a railway line and through a low-density residential area. The bridge has an overall length of roughly 265 m and a width of approximately 17 m.

The *Hoheluftbrücke* in its actual configuration is a reinforced concrete frame-bridge. It was newly built in 2009 in order to replace the existing 75 year old deficient two-span reinforced concrete bridge, which suffered from heavy spalling and leakages as well as from progressing reinforcement corrosion. The new bridge has a span of 17.20 m and an overall width of 46.36 m. It transfers the main traffic road Hoheluftchaussee across the narrow Grindelkanal channel. Although located directly in the urban part of Hamburg, the bridge is hardly visible to the local population because it is hidden by the vegetation along the channel banks.

The *Brücke Bahrenfelder Chaussee* transfers the main road Bahrenfelder Chaussee across the A7 motorway. It is designed as a prestressed concrete beam bridge with a box-girder cross-section. With its two spans of 28.01 m and 32.60 m it has an overall length of 60.61 m and a width of 32.50 m. It was built in 1970 and is going to be replaced in 2016 by an enclosure of the A7 motorway (Hamburg 2010b/c).

The *Hochstraße Elbmarsch* is the longest motorway bridge in Germany with an overall length of 3840 m. Designed by the technical office of the contractor Ed. Züblin, it was built in 1974 in order to transfer the A7 motorway in the form of an elevated motorway across the marshland south of the River Elbe. It consists of prefabricated prestressed concrete T-beams with an in-situ concrete slab. The girders are discontinuous with a regular span of 35 m and a regular width of 2×17.75 m (Waßmuth and Gass 1973). This functional structure with its outstanding technical dimensions is remotely located in the industrial surroundings of the port. In these surroundings it is rather driven on than looked at by the population, so the heavy appearance resulting from a low-cost design can be excused (Leonhardt 1984: 132–133)

The *Brennerhofbrücke* transfers the remotely located two-lane *Brennerhof* road across the A1 motorway. It is designed as a three-span steel-concrete composite deck, which rests on reinforced concrete piers. It has an overall length of about 60 m and a width of about 10 m. The bridge was built in 2006 in order to replace the existing bridge from 1962. The replacement became necessary because of the widening of the A1 motorway from four to six lanes (LSBG 2011). It is remotely located in the Elbe-grassland surrounded by some isolated industrial and commercial facilities. In spite of the relatively low traffic volume, the bridge is an important traffic link, connecting the districts Moorfleet, Ochsenwerder and Neudorf to the centre of Hamburg—a temporary closure of this bridge would lead to considerable traffic detour.

The *1. Peuter Brücke* is located in the exclusively industrial surroundings of the Port of Hamburg, where it serves as an important traffic link for the warehouses around the *Peutehafen* to the city centre and the Port of Hamburg. This steel bridge was built in 1914 as an arched truss. Is has a single-span of approximately 38 m and, with an overall width of about 16 m, it carries two lanes. With its elegant riveted arched truss the bridge fits well into the industrial surroundings. After 100 years in service, it was replaced in 2014 by a new steel-truss bridge (HC Hagemann 2014).

The *Brücke Wördemanns Weg* was built in 1961. It is a simply supported prestressed concrete bridge, which transfers a 1+1 lane road across the A7 motorway. Its overall length is 33.80 m and the width is 14 m. In its structural layout it is an in-situ concrete T-beam & slab bridge with prestressed girders. It is located in the outer district Stellingen in a low-density residential area. As it is the case for the Brücke Bahrenfelder Chaussee, this bridge will be replaced by an enclosure of the A7 motorway; the construction works will start in 2012 (Hamburg 2010b/c, 2014). The Brücke Wördermanns Weg is a functional structure without aesthetic specialities; it is well hidden to the local residents and only visible to the users of the motorway that runs beneath.

The *Brandenburger Brücke* is located in the industrial surroundings of the Port of Hamburg. The steel-truss superstructure was built in 1928. The bridge connects the office buildings of the port railway station, the Hafenbahnhof Hamburg-Süd, to the local road network. The traffic density on the bridge is very low because it is exclusively used by the people working in the office buildings. It has an overall length of about 36 m and—with just one traffic lane—a width of approximately 7 m.

### 3.2.3 Chennai Bridges

In Chennai altogether ten bridges were investigated. Seven of these bridges are maintained by the *Bridges Department* (CBD) of the Corporation of Chennai, which is responsible for all bridges belonging to the urban road network. The remaining three bridges are managed by the *Highways Department* (TNHD) of the Government of Tamil Nadu, which besides for the motorways is also responsible for the *Anna Salai* road in Chennai. This road is one of the most important north-south connections in Chennai since the seventeenth century (Hancock 2008: 26); it runs from the old British Fort St George in the North-East seven miles to the St Thomas Mount in the South-West of the city and is also well known by its former name *Mount Road*.

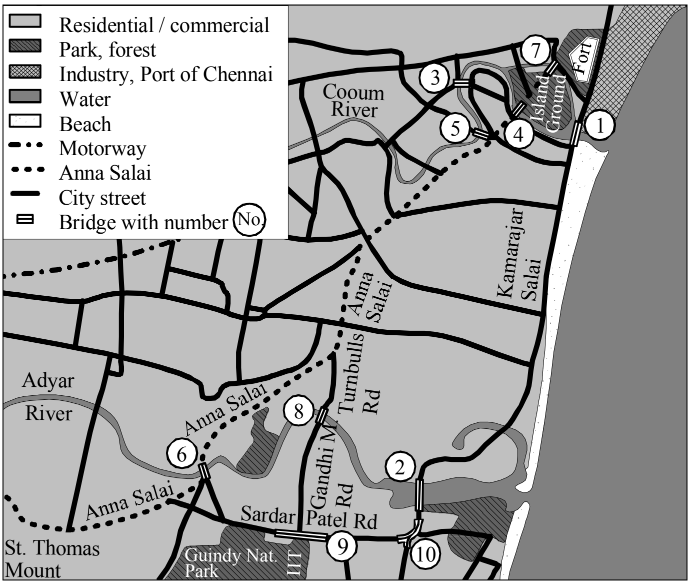



**Fig. 3.3:** Locations of the selected bridges in Chennai: (1) Napier Bridge, (2) Thiru Vi Ka Bridge, (3) St Andrew Bridge, (4) Periyar Bridge, (5) Harris Bridge, (6) Maraimalai Adigal Bridge, (7) Quaid-E-Millet Bridge, (8) Kotturpuram Bridge, (9) Sardar Patel Rd Flyover, (10) Adyar Flyover

In detail, the following ten bridges were investigated (Fig. 3.3 and Fig. 3.4):

- Napier Bridge (CBD)
- Thiru Vi Ka Bridge (CBD)
- St Andrew Bridge (CBD)
- Periyar Bridge (TNHD)
- Harris Bridge (CBD)
- Maraimalai Adigal Bridge (TNHD)
- Quaid-E-Millet Bridge (TNHD)
- Kotturpuram Bridge (CBD)
- Sardar Patel Rd Flyover (CBD)
- Adyar Flyover (CBD)

The *Napier Bridge* is a major landmark structure in Chennai. Built between 1932 and 1943 as a sequence of six bowstring concrete arches with a span of 24.47 m each, it is the first bridge in India that applied the then innovative concept of a concrete arch with a prestressed tension chord (Tappin 2003: 1935). This concept had been introduced shortly before in 1927 by Franz Dischinger for the construction of the Saalebrücke Alsleben (Herzog: 2005). The bridge was widened in the year 2000 to an overall width of approximately 2×12 m by the erection of an optically identical structure. It transfers the arterial Kamarajar Salai road across the mouth of the River Cooum and is perfectly embedded in the surrounding landscape. Together with the adjacent historic Senate House, which was built in 1879 by the Architect R. F. Chisholm as the first building of the of the University of Madras (Srinivaschari 1939: pp. xxxi–xxxiv and 262), the Napier Bridge forms an ensemble of heritage structures at the northern end of the Marina Beach. With its elegant low-rising white arches it wonderfully moulds into the low silhouette of the vegetation along the river banks and the adjacent Island Ground in the River Cooum. On the 20$^{th}$ of July 2010 the bridge was decorated with a permanent lighting arrangement of altogether 464 light bulbs, which illuminate the bridge after sunset (The Hindu 2010).

The monumental *Periyar Bridge* was built in 1805 with the name New St George's Bridge in order to replace the 1718 built St George's Bridge / Lord Willingdon Bridge (Love 1913/1996: 20, Srinivaschari 1939: 201, Parthasarathy 2013). The historic bridge consists of 11 masonry arches and has an overall length of 97.6 m. It was widened to a width of 24 m by a simple T-beam & slab bridge next to it. The Periyar bridge carries the arterial Anna Salai road with its 2×3 lanes across the River Cooum. With its pronounced arches and decorative pillars the historic side of the light painted bridge shows a grandiose appearance.

The historic *St Andrew Bridge* was built in 1815 under the direction of Lieutenant Colonel Thomas de Havilland of the Madras Corps of Engineers (Skempton 2002: 304). It is a three span bridge with an overall length of approximately 50 m whose arches were erected from cut stone within five month of time (Vibart 1883: 20–21). Crossing the River Cooum the bridge links the Chintadripet district—a former weavers' village founded in 1734 (Srinivaschari 1939: 148 and 335) and now a densely built-up area with low-rise residential housing—by a 2×1 lane carriageway with a width of approximately 12 m to the Egmore District with its regional railway station.

**Fig. 3.4:** Chennai Bridges (from left to right): *First row*: Napier Bridge, Periyar Bridge (from west, historic bridge), Periyar Bridge (from east, broadened with reinforced concrete girders); *Second row*: Harris Bridge (from north, historic bridge), Harris Bridge (from south, broadened with reinforced concrete girders), St Andrew Bridge; *Third row*: Thiru Vi Ka Bridge, Maraimalai Adigal Bridge, Quaid-E-Millet Bridge. *Fourth row*: Kotturpuram Bridge, Sardar Patel Rd Flyover, Adyar Flyover (Photos: Grischa Dette)

The historic *Harris Bridge* was built from 1851 to 1855 with three arches of approximately 16 m each crossing the River Cooum. Like the historic Periyar Bridge it has been widened by an additional T-beam & slab bridge to a width of approximately 18 m. The structure is important to traffic because it links the Egmore district to the arterial Anna Salai road, which constitutes the main north-south traffic route through Chennai.

The *Thiru Vi Ka Bridge* crosses the River Adyar next to the green compound of the Theosophical Society on the south bank and the residences of high ranking officials and the university campus on the north bank. The overall length of this major bridge is approximately

280 m with fourteen spans of about 20 m each, which consist of prefabricated concrete girders with an in situ concrete slab. The Thiru Vi Ka Bridge was built in 1973 as inscribed in the sockets of the characteristic lion statues decorating the bridge entrances. It was designed as a replacement for the historic Elphinstone Bridge, which was built during the governorship of Lord Elphinstone between 1837 and1842 (Hancock 2008: 95; Srinivasachari 1939: xii). The replacement became necessary because of the increasingly heavy traffic. The Thiru Vi Ka Bridge connects the new residential and industrial areas south of the River Adyar with the eastern artery road Kamarajar Salai, which runs tangentially to the city centre along the Marina Beach and passes eventually across the Napier Bridge in the north of Chennai. With its simple design and its low and clear silhouette it mirrors the clear horizontal and vertical lines of the distant high-rise buildings in the north-east while it clearly separates the irregular contour of the trees of the Theosophical Society from the smooth surface of the Adyar River.

The *Maramalai Adigal Bridge* is an important traffic link that transfers the Anna Salai (Mount Road) with 2×3 lanes across the Adyar River. In its current layout it is a twelve-span T-beam & slab bridge. The precast reinforced concrete girders have a span of 17.68 m each; the width of the bridge is approximately 30 m. In 1966 the Maramalai Adigal Bridge replaced the old Marmalong Bridge, which was built in 1762 by the Armenian merchant Coja Petrus Uscan (Srinivaschari 1939: 142). Today its surroundings on the northern bank of the Adyar River are characterised by the slum dwellings, which even extend under the bridge.

The *Quaid-E-Millet Bridge*, built in 1985, serves as the northern entrance to the busy Anna Salai road. It is a reinforced concrete structure with 2×3 lanes and a width of 21.9 m. In longitudinal direction it consists of three simply supported spans, which add to an overall length of 37.05 m. The bridge appears as a purely functional structure and is located at the north-eastern side of the Island Ground. To the north, south and east the bridge is embedded in the green vegetation of the Fort St George area and the Island Grounds, whereas to the east the location is characterised by slum dwellings along the river bank.

The *Kotturpuram Bridge* is located half way between the Maramalai Adigal Bridge and the Thiru Vi Ka Bridge. It was built in 1988 with three spans of approximately 40 m each as a T-beam & slab structure with prefabricated prestressed concrete girders. The bridge is an important traffic link because it connects the residential areas as well as the campuses of the Anna University and the IIT Madras to the northern centre of Chennai. With a width of approximately 20 m the bridge carries the 2×2 lane Turnbulls Road across the Adyar River. Located at a sharp bend of the Adyar River it is surrounded by overgrown riverbanks.

The *Sardar Patel Road Flyover* was built in 2000 in front of the IIT Madras campus and the Guindy National Park with the purpose of facilitating the traffic flow at the intersection of the Sardar Patel Road with the Gandhi Mandapam Road, which is an extension of the Turnbulls Road leading to the Kotturpuram Bridge. With an overall width of approximately 8 m it carries the east-west traffic of the Sardar Patel Road on two lanes across the road intersection. With fifteen spans it has an overall length of approximately 300 m. The structural layout is typical for this kind of flyover: Simply supported prefabricated prestressed girders rest on cross-beams on single columns. The contractor of the bridge was awarded the *Most Outstanding Bridge National Award-1999* from the Indian Institution of Bridge Engineers for its innovative construction engineering (Larsen & Toubro 2011).

The *Adyar Flyover* facilitates the traffic flow at the intersection of the Sardar Patel Road with the Lattice Bride Road and the Durgabai Deshmukh Road. With its Y-shaped layout that leads the traffic on 2 lanes from the Durgabai Deshmukh Road across the intersection to the two other streets it has an overall length of approximately 700m. Its general structural design resembles that of the Sardar Patel Road Flyover.

### 3.2.4 Survey Instruments – Questionnaire and Interviews

#### Questionnaire Layout

The questionnaire survey gathers information with respect to individual bridges, so that for each one of the survey bridges a separate questionnaire had to be answered. The questionnaire, which is contained in Appendix I of the present thesis, comprises five sections.

The first introductory section of the questionnaire asks for basic information regarding the bridge, such as its name, size, age, as well as its location. This basic information is not subject to further data analysis. The main purpose of acquiring these data is simply to provide a basis for interpreting the data that are gathered by the further questionnaire sections.

The second section of the questionnaire aims at gathering data from which to identify bridge specific stakeholders' demands. As it is explained in chapter 2, these stakeholders' demands are theoretical constructs that describe the preferences of the different parties—namely the users, local residents, business, and society as a whole—with respect to the in-service behaviour of the bridge. Generally, these stakeholders' demands can be categorised into four groups: proper functioning, economy, society & culture, and environment. However, the stakeholders' preferences regarding economic aspects are not investigated in the scope of the present investigation. This is because economic aspects in terms of maintenance costs are directly carried by the bridge owning authority, whereas the stakeholders— users, local residents, business, and society—are usually not interested in bridge specific maintenance budget allocation—notwithstanding exceptional cases, such as those of major bridges with high maintenance expenditures, where also the maintenance costs may become an issue of major public interest. On the basis of analogous considerations, also environmental aspects are not covered by the actual survey.

A straight forward way to investigate these stakeholders' demands would be direct questioning of the users and the otherwise affected people with the help of a through survey study. However, such a time and work intensive investigation is not feasible—not in the scope of the present thesis and expectably also not in practical maintenance planning. Therefore, a set of qualitative criteria, which can be judged by the bridge manager and the interviewer, has been defined in order to infer the stakeholders' demands (Table 3.1).

**Table 3.1:** Criteria for identifying the stakeholders' demands

| Topic | Stakeholders' demands | Criteria |
|---|---|---|
| Function | Traffic availability<br>(Demand: The traffic flow on the bridge is not restricted or impaired.) | - Traffic volume<br>- Importance for network<br>- Possible stress on alternative routes<br>- Traveller detour |
| Society | Aesthetic appearance<br>(Demand: Sound visual appearance because of a limited number of visible defects or dirty surfaces.) | - Viewing distance<br>- Viewing time<br>- Surroundings<br>- Interest of local residents /general public |
| Culture | Preservation / service life<br>(Demand: Preserve structures of cultural interest; replacement should be avoided.) | - Technical masterpiece<br>- Symbol / artistic value<br>- Integration into environment<br>- Historic value |

All items are of the multiple choice type—each with five options that represent ascending intensity levels. For example, the options range from a '*negligible*' to a '*very high*' traffic flow on the bridge or from the agreement levels '*is not true*' to '*perfectly applies*' with respect to the question for an *extraordinary historic value or scarcity value*.

The responses to the multiple choice questions are prepared for further data analysis with the help of a 1 to 5 scale. As a first step, the answers to the individual items are transferred to values between 1 and 5. As a second step, a resulting score for each one of the three aspects of the stakeholders' demands is calculated from the ratings of all items that represent each demand-aspect. In the case of traffic availability and aesthetic appearance it is calculated as the mean value of the item ratings, while in the case of the service life the maximum item rating is taken as the representative value. In the latter case, this is because each one of the criteria that are used by the questionnaire in order to identify the cultural importance of the bridge can be decisive alone.

An example for this is provided by the Table 3.2. Finally, this procedure results in a bridge specific combination of ratings for three aspects—service life, appearance, and traffic availability.

**Table 3.2:** Example data preparation for the functional aspect *traffic availability*

| Criterion | Answer | Rating [a] |
|---|---|---|
| The **traffic volume on the bridge** is (...) | *low* | 2 |
| The **traffic volume in the surrounding** road/rail **network** is (...) | *medium* | 3 |
| The **importance of the bridge for the surrounding network** is (...) | *high* | 4 |
| In case of (partial) closure, **the stress on alternative routes** is (...) | *very high* | 5 |
| The **detour** in case of (partial) closure is (...) | *very high* | 5 |
| Resulting rating (= mean value) | ≈ *high* | 3.8 |

[a] Rating scale transformation: *negligible*: 1, *low*: 2, *medium*: 3, *high*: 4, *very high*: 5

The third and the fourth section of the questionnaire are designed in order to provide data from which to identify the bridge specific *owner's requirements* and *maintenance objectives*. In contrast to the indirectly questioned stakeholders' demands, these two constructs are measured easily by single items —simply by asking about the bridge manager's judgement about the respective importance. These single item representations are believed to be a sufficient source of information because the responsible bridge manager usually has an idea about the requirements for the bridge and his maintenance objectives.

The fifth questionnaire section contains questions for identifying characteristic patterns of the applied maintenance strategy. As it has been described above in sub-section 2.6.3, maintenance strategies can be classified in terms of their proactive/reactive as well as their preventive characteristics. In order to identify these characteristics, the questionnaire questions do not directly ask for the *proactive* or the *preventive* bias—terms that may be misleading to the questioned person—but they rather refer to certain criteria from which the proactive and preventive orientation can be inferred (Table 3.3). First, the items whose purpose is to identify the proactive bias by asking for the deterioration level at the time of maintenance application refer both to the structural elements (super- and substructure) and to the bridge equipment, such as expansion joints and drainage system. This is done in order to identify strategies that are characterised by a selective proactive maintenance bias with respect to structure and equipment. Second, the items whose purpose is to identify the preventive bias ask for the application practice with respect to special preventive activities and for the intensity of routine minor maintenance.

**Table 3.3:** Strategy characteristics and associated criteria

| **Strategy aspects** | **Criteria** |
|---|---|
| *Proactive/reactive bias* | |
| Unspecific | General deterioration level at the time of repair |
| Structure | Deterioration level at the time of repair (structural elements) |
| Equipment | Deterioration level at the time of repair (equipment parts) |
| *Preventive bias* | |
| Special activities | Selection of activities that reduce future deterioration |
| | Application of special preventive measures (e.g. coating) |
| | Frequency of special preventive activities |
| Routine maintenance | Intensity of routine minor maintenance |

The items of this questionnaire section are also of the multiple choice type. Most of the items allow selecting from a list of five possible answers—with the exception of the items that specifically refer to the frequency of preventive activities and intensified routine maintenance; these apply a list of only four possible answers. This is because during the questionnaire design it occurred that more than four different levels for the specification of the frequency lead to hardly distinguishable frequency descriptions.

The preparation of the data from the fifth questionnaire section on maintenance necessitates elaborate data aggregation. This is because the *proactive* and the *preventive* bias are indirectly asked for with the help of several items (Table 3.4).

**Table 3.4:** Items for strategy description

| Item code | Item Topic | Rating scale |
|---|---|---|
| *Condition (deterioration level) at the time of intervention* | | 1 – 2 – 3 – 4 – 5 [a] |
| $det_0$ | Overall deterioration | |
| $det_1$ | Concrete repair (superstructure) | |
| $det_2$ | Concrete repair (substructure) | |
| $det_3$ | Rehabilitation of tendons | |
| $det_4$ | Rehabilitation of kerbs | |
| $det_5$ | Rehabilitation of asphalt pavement/surfacing | |
| $det_6$ | Rehabilitation of drainage inlets | |
| $det_7$ | Rehabilitation of drainage pipes | |
| $det_8$ | Rehabilitation of expansion joints | |
| $det_9$ | Rehabilitation of bearings | |
| $det_{10}$ | Rehabilitation of the steel structure | |
| $det_{11}$ | Rehabilitation of the corrosion protection paint | |
| $det_{12}$ | Replacement of the bridge or essential parts | |
| *Application of preventive activities* | | |
| $prev_1$ | Activities which delay deterioration | 1 – 2 – 3 – 4 – 5 [b] |
| $prev_2$ | Special preventive measures | 1 – 2 – 3 – 4 – 5 [b] |
| $prev_3$ | Treatment of concrete surface | 1 – 2 – 4 – 5 [c] |
| $prev_4$ | Corrosion protection of Steel (e.g. paining) | 1 – 2 – 4 – 5 [c] |
| *Intensity of routine minor maintenance* | | |
| $routine_1$ | Application of routine minor maintenance | 1 – 2 – 4 – 5 [c] |

[a] All items referring to the deterioration level are rated on a scale in the range from 1 (critical condition) to 5 (satisfactory / good condition).

[b] These items refer to the general application of preventive maintenance activities and are rated on a scale in the range from 1(*never / does not apply*) to 5 (*usually / perfectly applies*).

[c] These items are rated on a 4-level scale. For data aggregation the rating score for these items is defined by leaving out a rating of 3 as follows: 1 (*never*), 2 (*infrequently*), 4 (*regular),* 5 (*intensive*).

The data preparation for further analysis is achieved by averaging the item ratings. First, the *proactive bias with respect to the bridge structure* is derived from the degree of deterioration of the main structure (equation 3.1). In this case, the relevant items refer to the deterioration of the concrete surface of the super- and the substructure as well as the deterioration of the tendons and the kerbs (items $det_1$ to $det_4$). Also the items that refer to heavy interventions regarding the steel structure ($det_{10}$) or to the replacement of the whole bridge ($det_{12}$) are relevant in this context. Second, the *proactive bias with respect to the equipment* is derived from the deterioration level of the bridge equipment at the time of intervention (equation 3.2). The items, which are relevant in this context, refer to the condition of the carriageway and the drainage system as well as that of the bearings and the expansion joints (items $det_5$ to $det_9$). Third, an indication for the *preventive bias* is derived from all the items, which refer to typically preventive aspects (items $prev_1$ to $prev_4$). Also the item $det_{11}$ is included—it refers to the condition of the steel paining at the time of rehabilitation and thus indicates the intensity of preventive repainting (equation 3.3). Finally, a second indication for the preventive bias, the intensity of *routine maintenance*, is defined by equation 3.4 and is derived from a single item ($routine_1$). In both cases—concerning the proactive and the preventive bias—the boundaries between proactive and (non-proactive) repair and between preventive and non-preventive characteristics are defined by a rating of '3.0'. In the case of the proactive bias this represents intervention at a condition that is not worse than 'poor' and in the case of the preventive bias this stands for regular application of special preventive activities and regular minor maintenance.

$$\textit{proactive bias (structure)} = \frac{1}{7}\left(det_0 + det_1 + det_2 + det_3 + det_4 + det_{10} + det_{12}\right) \tag{3.1}$$

$$\textit{proactive bias (equipment)} = \frac{1}{6}\left(det_0 + det_5 + det_6 + det_7 + det_8 + det_9\right) \tag{3.2}$$

$$\textit{preventive bias (special activities)} = \frac{1}{5}\left(prev_1 + prev_2 + prev_3 + prev_4 + det_{11}\right) \tag{3.3}$$

$$\textit{preventive bias (routine maintenance)} = routine_1 \tag{3.4}$$

with

$$\begin{array}{lll}
\textit{proactive bias (structure}\text{ or }\textit{equipment)} \geq 3.0 & \rightarrow & \text{proactive} \\
\textit{proactive bias (structure}\text{ or }\textit{equipment)} < 3.0 & \rightarrow & [-] \\
\textit{preventive bias (special activities}\text{ or }\textit{routine maint.)} \geq 3.0 & \rightarrow & \text{preventive} \\
\textit{preventive bias (special activities}\text{ or }\textit{routine maint.)} < 3.0 & \rightarrow & [-]
\end{array}$$

**Interview**

The interview gathers information on the general maintenance planning processes and covers three topics (Table 3.5): First, the interview deals with the general framework of maintenance planning, then the focus shifts to practical maintenance planning and scheduling. Finally, the interview questions refer to the assessment and evaluation of maintenance strategies.

The first block of questions investigates the *general framework of maintenance planning* by asking for the general strategy approaches that are applied by the interviewee and by seeking information concerning the objectives that are considered for maintenance planning. Also the relative ranking of the considered objectives is of special interest within this first interview topic. Furthermore, the impact of budgeting on maintenance planning decisions is asked for in order to identify the actual relevance of the economic objective. The latter question is of particular interest because the economic objective is one of the three pillars of the traditional strategy assessment paradigm—next to the aspects of structural safety and overall bridge condition—as it is described in the above section 2.7.

**Table 3.5:** Interview topics

| **Topic** | **Subject of interview question** |
|---|---|
| *General framework of maintenance planning* | |
| General strategy | General description of the maintenance strategy |
| Objectives | Specific objectives (bridge stock / individual bridges) |
| | Ranking of objectives |
| Impact of budget | Impact of budgeting on maintenance planning decisions |
| *Maintenance planning and scheduling practice* | |
| Selection | Criteria for selection of maintenance activities |
| Prioritisation | Prioritisation of bridges for maintenance as well as prioritisation of maintenance activities for a certain bridge |
| Scheduling | Maintenance planning and scheduling practice |
| *Assessment and evaluation of maintenance strategies* | |
| Retrospective | Application of retrospective assessment / evaluation |
| | Application of indicators, such as bridge condition |
| Future maintenance | Comparison of strategy alternatives (e.g. variation studies) |
| | Performance indicators and optimisation techniques |

The *maintenance planning and scheduling practice* is the topic of the second question block. One theme of this block is the question for the criteria which the bridge managers apply for selecting specific maintenance options, for example on which basis they choose a certain maintenance measure from all possible maintenance solutions for a given defect. Further questions focus on prioritisation of maintenance actions. Here the term prioritisation refers both to bridges that are maintained prior to other bridges and to maintenance activities that are applied prior to other activities. Also the criteria that are applied for scheduling maintenance activities are explored in this context.

Finally, the third block of questions explores to what extent the bridge managers apply methods for the *assessment and evaluation of maintenance strategies.* A first issue in this context is the question if the bridge manager applies methodologies for the retrospective assessment and evaluation of maintenance strategies—and if so, which actual performance indicators are applied for this purpose. A second issue concerns the approaches to the strategic planning of future maintenance works. Specific questions in this context are the application of performance indicators and possibly of optimisation techniques.

## 3.3 Bridge Specific Demands, Requirements, and Maintenance Objectives

### 3.3.1 Introduction

It is a central assumption of the present thesis that the preferences of the stakeholders, of the owner, and of the bridge manager depend on the bridge specific context. It is central because this assumption implicates the need for the development of an approach that allows for bridge specific and multi-objective assessment and evaluation of maintenance strategies. The present sub-section seeks to provide an empiric indication for this claimed hypothesis by conducting the following investigation:

- As a first step, five different bridge categories are defined, which represent characteristic contexts, such as the urban integration or the importance to the traffic network (Table 3.6).
- As a second step, graphic preference profiles are generated from the questionnaire data at all three stages—demands, requirements, objectives—in the form of *star plots* with the four axes representing the aspects *service life*, *aesthetic appearance*, *costs*, and *traffic availability*.
- Then, as a third step, it is investigated, if the preference profiles of the bridges show a typical pattern for the defined categories. If this turns out to be the case, empiric evidence is found for the need to consider the bridge specific context for the assessment and evaluation of maintenance strategies and for strategic maintenance planning.

In addition to this, the responses to the first block of interview questions are used to provide a further foundation for the claimed hypothesis. For one thing the answers to the questions regarding the maintenance objectives and the impact of budgeting, of which the latter pays particular attention to the role of the economic objective, are used as a reference basis for interpreting the results from the questionnaire survey. For another thing the interview responses also contribute to a deeper understanding of how bridge managers set maintenance objectives and to what extent they consider stakeholders' demands and bridge specific situations in their strategic planning decisions.

**Table 3.6:** Bridge categories

| Bridge category | General description of the individual characteristics |
|---|---|
| Landmark bridge | Impressive major bridge, usually a commonly agreed point of interest [a] |
| Heritage bridge | Bridge of historic or other cultural importance [b] |
| City bridge | Bridge located in urban surroundings (e.g. a flyover or a road bridge across a river). [c] |
| Remote main link | Bridge located outside of urban surroundings but important for traffic (e.g. a bridge belonging to a motorway outside the city). |
| Remote minor link | Bridge located outside of urban surroundings and usually of small dimensions and minor importance for the traffic. |

[a] Bridges that show characteristics of *landmarks* as well as of any other bridge category are allocated to the category *landmark*.

[b] Bridges that show characteristics of *heritage bridges* as well any other bridge category except *landmark bridge* are allocated to the category *heritage bridges*.

[c] *City bridges* are usually of small/medium size as opposed to the category *landmark bridge* and they do not fulfil characteristics of *heritage bridges*.

The data collection of the questionnaire and interview survey together with the detailed data analysis can be found in the in an unpublished data collection and analysis at the Institute of Concrete Structures of the Hamburg University of Technology (Dette 2014a).

### 3.3.2 Data Analysis

#### Hamburg

The 14 investigated bridges in Hamburg are allocated to the five categories as listed in Table 3.7:

**Table 3.7:** Allocation of the Hamburg bridges to categories

| Bridge category | Bridges in Hamburg | Quantity |
|---|---|---|
| Landmark bridge | Brücke K6 Moorfleet, Freihafen Elbbrücke, Köhlbrandbrücke | 3 |
| Heritage bridge | Zollenbrücke, Lombardsbrücke, Kennedybrücke | 3 |
| City bridge | Robert-Schumann Brücke, Hoheluftbrücke, Brücke Bahrenfelder Chaussee | 3 |
| Remote main link | Hochstraße Elbmarsch, Brennerhofbrücke, 1. Peuter Brücke | 3 |
| Remote minor link | Brücke Wördemanns Weg, Brandenburger Brücke | 2 |

For each bridge category the preference profiles are depicted in a star plot diagram (see Fig. 3.5).

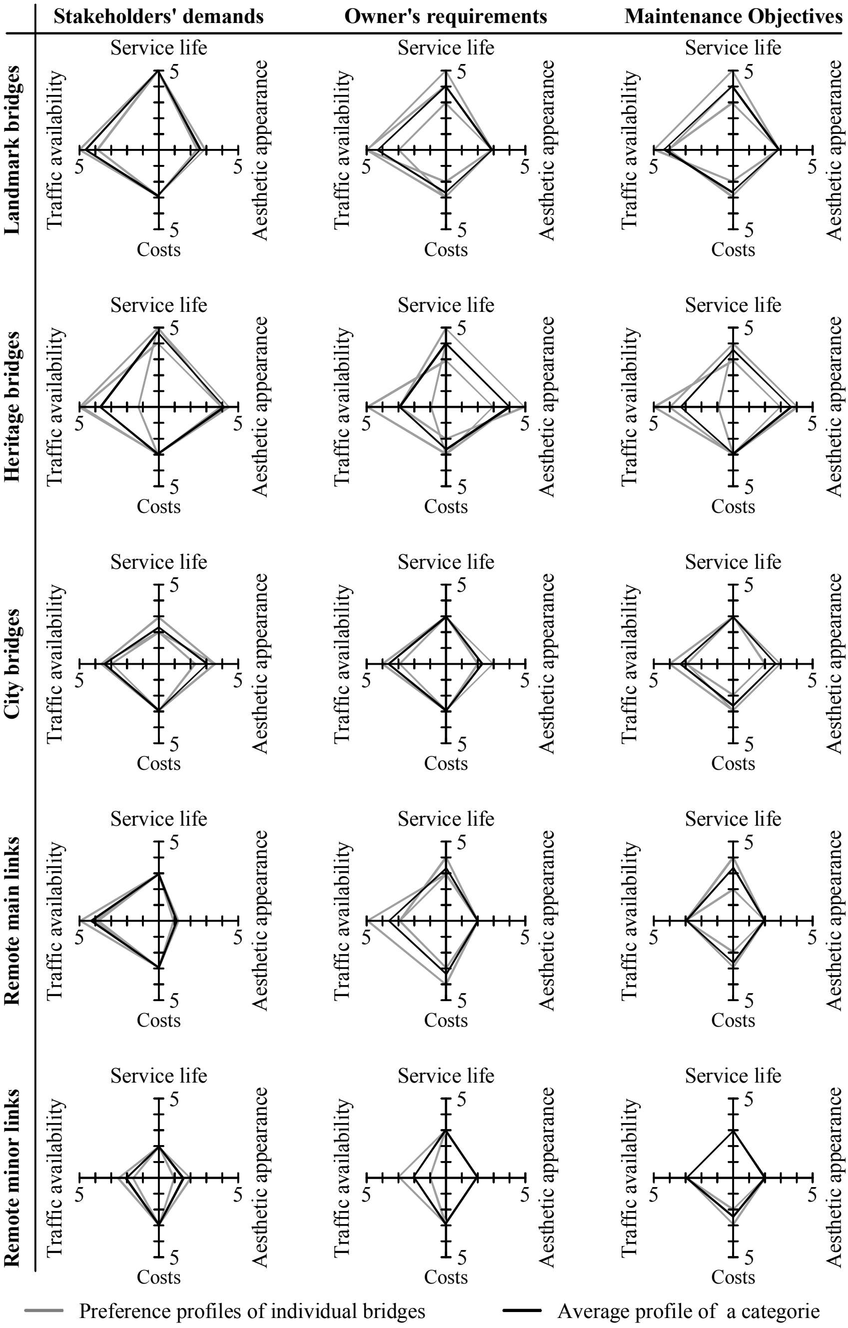



**Fig. 3.5:** Preference profiles for the five bridge categories: The grey lines show the profiles of individual bridges, whereas the black lines represent the mean preference profiles for each category. The stakeholders' demands with respect to the aspect *costs* are set uniformly to a value of '3'. (Data source: Dette 2014a)

From the above Fig. 3.5 it can be seen that the preference profiles of each category show a characteristic pattern that can be distinguished from category to category. The comparison shows that the characteristic profiles differ most clearly in terms of the stakeholders' demands and also, to a lesser extent, with respect to the owner's requirements. Also maintenance objective pattern are still recognizable, though to a reduced extent.

At the basic level of the stakeholders' demands, the profiles show distinctive patterns with respect to the relative valuations of the aspects availability, service life, and appearance. Only the aspect 'costs' does not vary because it is uniformly set to a rating of '3' in order to ease comparability between the profiles of the stakeholders' demands, the owner's requirements, and the maintenance objectives. The *landmark bridges* both excel with respect to the aspects availability and service life while they only show a medium score as far as the importance of the aesthetic appearance is concerned. The *heritage bridges* also show an absolutely high valuation of the service life, but unlike the landmarks, the second clear focus is rather on appearance than on availability; the ratings with respect to availability vary from very low (Zollenbrücke) to very high (Kennedybrücke). In contrast, the *city bridges*, as it appears from the deltoid shape of their profiles, are characterised by a comparatively high valuation of availability and appearance—the two aspects which are valuated so differently for landmarks and heritage bridges—while they show only a below-medium importance of preservation. The *remote main links* show the second highest valuation of availability in absolute terms, while they show only medium valuation of service life and, together with the remote minor links, the lowest rating for appearance. In some way, the profile is similar to that of the landmarks, as it highly values availability and service life while the appearance seems comparatively unimportant—however, a major difference to the landmark bridges is the exclusive focus on availability with service becoming second by a large margin. Finally, in the case of the *remote minor links* no clear focus on one of the three aspects service life, appearance, or traffic availability can be found. However, it is noteworthy that these bridges show the lowest ratings compared to all other bridges—with the only exception of the main traffic links, which score similarly low with respect to the aesthetic performance. The comparatively high rating of the economic domain is simply due to the predefined rating of '3' at this stage.

The characteristic profiles of the owner's requirements mostly resemble the profiles of the stakeholders' demands, though they are usually less pointed and show more scatter between the bridges of the same category. Generally, the absolute ratings change from more extreme valuations at the demands-stage ($\leq 2$ or $\geq 4$) to more medium ratings; on the other hand the relative importance of the aspects within the categories stays stable in most cases. However, two noteworthy shifts in the hierarchy can be found, which occur for the city bridges and the remote minor links. For one thing, the city bridge profiles, which are characterised by equal relevance of the aspects availability and appearance on the stakeholder-level, shift to a clear focus on availability, while the aspects service life and costs are both second and the formerly important appearance now becomes the least important requirement. For another thing, the remote minor links obtain an additional important owner-requirement by the introduction of the aspect costs, which now forms together with service life the major focus of interest in favour of the less important appearance. In the case of the remote main links the aspect costs becomes as important as the service life, while the appearance stays the least important point.

The profiles of the maintenance objectives resemble the profiles of the owner's requirements. However, the objective profiles become even less pointed and appear more and more alike. Once more, the absolute ratings tend to medium ratings, while the hierarchy of the aspects within the categories basically stays stable.

In their responses to the interview questions regarding the objectives of maintenance planning and their comparative importance, the bridge managers of both authorities mention the aspects service life/durability, traffic safety and costs. One adds the aspect traffic capacity, while the other adds the aspect structural safety. With respect to the ranking of these objectives, both bridge managers agree in terms of the lesser importance of the costs for the strategic maintenance planning. This is illustrated by the answers given to the question regarding the interdependency between maintenance planning and budget. Even if both authorities have only limited maintenance budgets to their disposal, cost minimisation is not applied during maintenance planning. Instead, less urgent measures are deferred in case the estimated total costs for the planned maintenance exceed the yearly budget. Furthermore, both agree in their valuation of the aspect service life/durability as medium important. Nevertheless, the bridge managers differ in their perception of the most relevant aspects; one mentions both traffic safety and traffic capacity, whereas the other considers structural safety to be of prime importance and assigns to traffic safety only medium relevance. Besides, none of the interviewed decision makers mentions the aspect aesthetic appearance.

However, bridge specific grading of the preferences is only mentioned with respect to traffic performance. In this context, one manager mentions that the aspect traffic capacity is playing a particular role for bridges along main traffic routes. They are checked in short intervals during regular *Streckenkontrollen* (road inspections) in addition to the routine bridge inspections. The recommended or prescribed intervals of these road inspections depend on the road category (e.g. motorway), the traffic importance and the average traffic flow. This is common practice in road management (Rosauer 2010: 16–17) with the objective to perform interventions very early and thus avoid major traffic obstructions.

**Chennai**

The 10 investigated bridges in Chennai are allocated to the different categories as listed in Table 3.8.

**Table 3.8:** Allocation of the Chennai bridges to categories

| Bridge category | Bridges in Chennai | Quantity |
|---|---|---|
| Landmark bridge | Napier Bridge, Periyar Bridge | 2 |
| Heritage bridge | Harris Bridge, St Andrew Bridge | 3 |
| City bridge | Thiru Vi Ka Bridge, Maraimalai Adigal Bridge, Quaid-E-Millet Bridge, Kotturpuram Bridge, Sardar Patel Road Flyover, Adyar Flyover | 5 |

The preference profiles for the bridges of the three different categories—city bridge, landmark bridge and heritage bridge—are depicted in Fig. 3.6 together with their resulting mean profiles. As it is the case with the Hamburg bridges, the aspect *costs* is set uniformly to the medium value '3' at the stage of the stakeholders' demands.

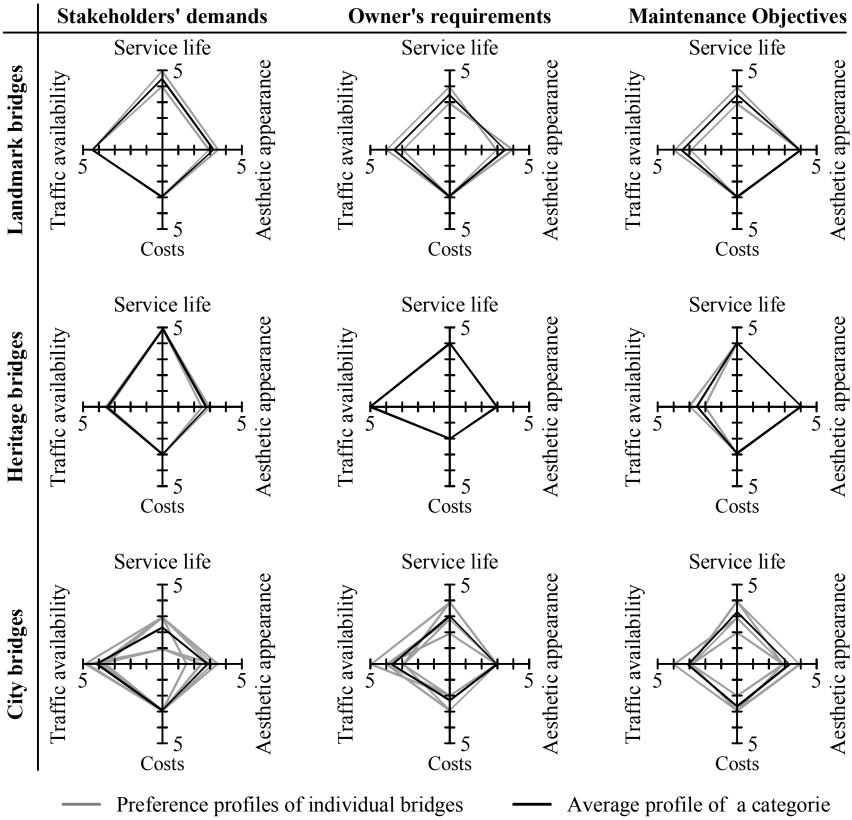



**Fig. 3.6:** Preference profiles in the form of star-plots for individual bridges from the three different bridge categories (city bridge, landmark bridge, and heritage bridge)—the dashed lines show the profile of individual bridges, whereas the solid lines represent the mean preference profiles for each category. (Data source: Dette 2014a)

The bridge specific preference profiles of the above Fig. 3.6 also show patterns, which can be distinguished from category to category. However, the differences are less clear than it is the case with the Hamburg bridges.

The stakeholders' demands profiles of each category show characteristic properties: The *landmarks* and *heritage bridges* both show a focus on service life, while they exhibit only a medium score as far as the importance of the aesthetic appearance is concerned. In the case of the landmarks, the availability turns out to be equally important as the service life, whereas in the case of the heritage bridges this aspect is only second to service life and only slightly more important than the aesthetic appearance. The main point of interest in the category *city bridges*, by contrast, is the availability, while service life seems to be least important and the appearance lies in between these two aspects.

Also the profiles of the owner's requirements turn out to be characteristic for the three bridge categories. However, it is also found from Fig. 3.6 that they reveal some remarkable differences to the profiles of the stakeholders' demands. In the case of the *landmark bridges*, there is a considerable shift in the importance of traffic availability and service life. By now they are considered to be only equally important as costs and appearance, as it occurs from the almost quadratic shape of the preference profiles. In the case of the *heritage bridges*, for which the performance aspects are rated exactly the same for both bridges, as it occurs from the single profile in Fig. 3.6, the traffic availability is rated much higher than it should be expected from the stakeholders' demands profiles. With the high ratings of service life and availability in combination with the low ratings both for costs and appearance, it rather resembles the stakeholders' demands profile for landmark bridges. Finally, the profiles for the *city bridges* reveal an increased valuation of service life. Remarkable in all cases is the fact that the lowest importance is attributed to costs; in every category this aspect is ranked below-average. Another noteworthy finding is the fact that all bridges investigated in Chennai are rated '3.0' on the appearance scale with the only exception of the Napier Bridge, where the appearance is judged to be very important (4.0).

The profiles of the maintenance objectives reveal a different picture in such a way that they become difficult to distinguish between the categories *landmark bridges* and the *city bridges*, which both exhibit more or less quadratic profiles. However, the two *heritage bridges* turn out to possess characteristic maintenance objective profiles with bias on service life and aesthetic appearance. Remarkably, it can be observed that, as a general trend, the aesthetic performance is rated higher while the traffic availability is rated lower across all bridge categories, when compared to the previous level of the owner's requirements.

From the responses to the interview questions, the main objective of maintenance planning seems to be for the two authorities to keep each bridge in good—or sufficient—condition by regular inspection and quick repair of the detected defects. Beneath that objective, only traffic availability seems to be of some concern. For example, one bridge manger points out, that bridges along so called *VIP-roads*, which are very important main traffic roads such as the Anna Salai in Chennai, are maintained with high priority of traffic aspects. However, the aspect traffic is only mentioned in connection with the actual selection and prioritisation of maintenance measures; thus, the availability of the bridge for traffic seems to only of concern on the operational level and not to be of strategic interest for the overall long-term maintenance planning. The costs do not seem to be of strategic concern either. Even if in the case of one authority the budget is fixed and maintenance measures may have to be deferred, the bridge manager does not valuate cost as determining maintenance planning. Also, the bridge manager of the other authority claims that it had never happened that the proposed maintenance plan, which is based on pure technical considerations, could not be implemented because of budget constraints. Only during operational maintenance selection costs are said to be given some importance.

## 3.4 Bridge Specific Maintenance Practice

### 3.4.1 Introduction

After gaining insight into the role that the bridge specific context plays in the definition of performance requirements and objectives, it remains to find out its role in maintenance planning decisions. For this purpose the following investigation is conducted:

- As a first step, graphic strategy profiles in the form of *star plots* are generated from the data of the fifth questionnaire section. The four axes of these diagrams represent the strategy characteristics *proactive bias (structure)*, *proactive bias (equipment)*, *preventive bias*, and *routine maintenance*. In doing so, the strategies of the bridges can be compared on the basis of their proactive and preventive characteristics.
- As a second step, it is investigated, if the strategy profiles of the bridges show typical pattern for the defined categories. If this turns out to be the case, empiric evidence is found for bridge specific diversification of maintenance strategies.

In addition to this, a more specific look is taken on the actual decision-making-process that eventually leads to bridge specific maintenance application patterns. This investigation of the *maintenance planning and scheduling practice* relies on interviews with the bridge managers and explores the criteria that are applied for maintenance activity selection and intervention scheduling. The aim of this investigation is to find out to which extent the bridge specific situation actually influences maintenance planning processes and if there is potential room for improvement by means of new systematic approaches yet to be developed.

### 3.4.2 Data Analysis

#### Hamburg Bridges

The strategy profiles that describe the *proactive* and *preventive* bias of the applied maintenance strategies are displayed in the following Fig. 3.7.

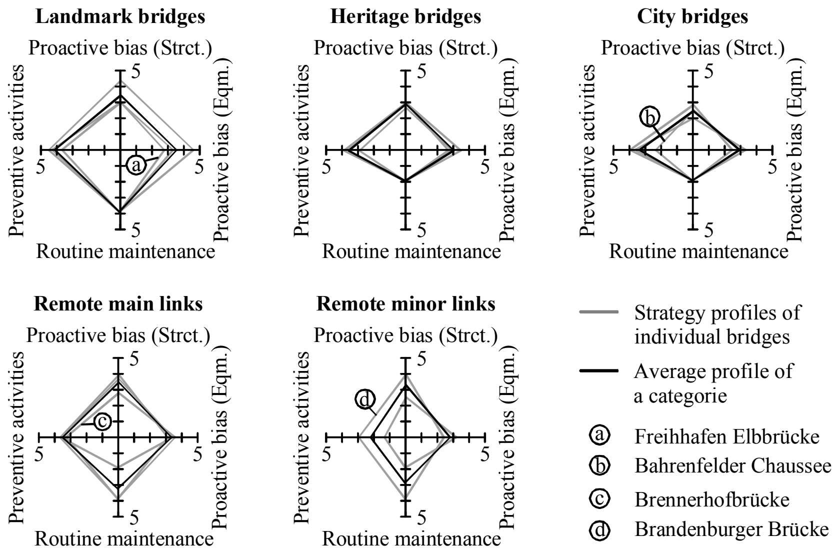



**Fig. 3.7:** Strategy profiles of the Hamburg bridges with respect to the maintenance application characteristics; *Strct.*: structural, *Eqm.*: equipment. (Data source: Dette 2014a)

From the above Fig. 3.7 it occurs that the maintenance application characteristics belonging to the categories landmark bridges and remote main link both show a clear *preventive-proactive* bias: First, with the only exception of the *Brennerhofbrücke*, which is classified as a remote main link, minor maintenance works are regularly applied. This is expressed by a rating of 4.0 for the indicator *routine maintenance* in the above Fig. 3.7. Second, the 'preventive activities'-indicator is well above 3.0 for all bridges of these two categories. And finally, the two proactive indicators are found to be also 3.0 or above, with the only exception of the landmark *Freihafen Elbbrücke*, where the expansion joints have been maintained only at serious condition.

Also the strategies that are applied to the three heritage bridges show *preventive-proactive* aspects. However, with indicator ratings for the proactive bias of only 3.0, infrequent minor maintenance works and a relatively high rating of around 3.5 for the application of preventive activities, these strategies are close to be of the *preventive-repair* type. This strategy type, in fact, is found to be applied to the city bridges, as can be seen from the profiles in the above Fig. 3.7. Only the *Brücke Bahrenfelder Chaussee* shows a very low rating for the indicator preventive activities, even in spite of the fact that concrete coating has been applied during major repair works 18 years after construction. However, the low preventive and proactive ratings for this particular bridge can possibly be explained by the planned replacement of this bridge in 2016 during the works for the enclosure of the A7 motorway (Hamburg 2010b/c).

In the case of the remote minor links, the *Brücke Wördemanns Weg* is clearly maintained according to a *reactive repair strategy*, while the strategy for the more than 80 years old steel bridge *Brandenburger Brücke* is rather of the *preventive-repair* type.

The actual *maintenance planning and scheduling practice* is investigated by considering the bridge managers' actual decisions in terms of *activity selection*, *prioritisation* and *scheduling*. As far as *activity selection* is concerned, one of the Hamburg bridge managers mentions the application of concrete coatings—possibly after preparatory concrete surface repair—for visual reasons; such a surface treatment can yield a smooth and attractive surface. Another activity, which is selected on a regular basis for all steel bridges, is the regular renewal of corrosion protection system of steel bridges. With respect to *prioritisation* of bridges, both authorities provide preferential treatment of bridges with relatively high traffic importance. An additional aspect mentioned by both bridge mangers is the special treatment of bridges, which are either affected by higher level development planning or subject to actual major construction works, such as the widening of motorways. In this case, maintenance activities may be deferred due to the fact that the bridge is planned to be replaced together with future works anyway. Activities may also be preponed in order to take advantage of synergy effects. The *scheduling* of maintenance activities is done by both authorities under consideration of traffic effects. For example, Maintenance works are preferably conducted during summer season, at weekends and during the night due to less commercial traffic. Furthermore, technical, economic and climatic aspects have a significant influence on the scheduling of maintenance measures. An example for this is that works on corrosion protection, for instance, should be avoided in the winter season because the temporary protection measures cause significant additional costs. One more aspect influencing the sequence of works is the recommended renewal of the asphalt pavement prior to rehabilitation of the bridge soffit. By this the upper side of the bridge is sealed anew and the restored soffit is not immediately subject to further deterioration.

**Chennai Bridges**

The strategy profiles that describe the characteristics of the applied maintenance strategies are displayed in the following Fig. 3.8.

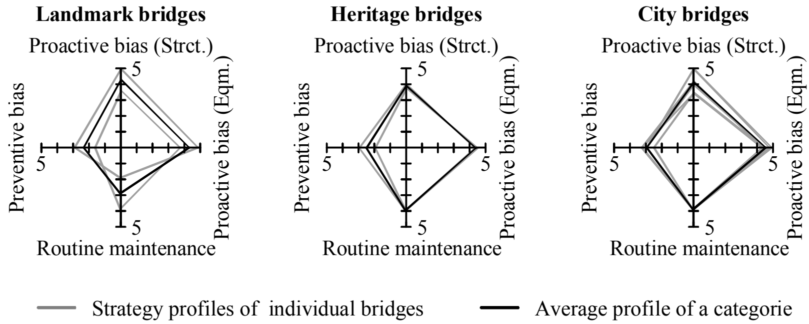



**Fig. 3.8:** Strategy profiles of the Chennai bridges with respect to the maintenance application characteristics; *Strct.*: structural, *Eqm.*: equipment. (Data source: Dette 2014a)

From Fig. 3.8 no considerable differences can be found between the strategy profiles of the bridge categories. Instead, the profiles suggest a commonly applied *proactive repair strategy* with some preventive aspects. The very low deterioration levels of the structure and the equipment at the time of intervention fulfil the proactive criteria, while the infrequent application of typical preventive measures is not sufficient to speak of preventive strategies—even if regular minor maintenance is being intensively applied.

During the interviews, the bridge managers of both Chennai authorities describe their general maintenance planning and scheduling practice in such a way that interventions are conducted '*as soon as possible*' whenever defects are noticed during the annual bridge inspections. This is a typical proactive approach. An exception to this is found for steel structures, where the general maintenance strategies show preventive aspects in terms of a routine biannual paining for corrosion protection, as one of the interviewed bridge managers explains.

Regarding the *selection* of maintenance measures, one bridge manager stresses that his bridge authority considers many aspects: condition, costs, traffic effects, long term effects and sometimes even aesthetic effects, as in the case of the Napier Bridge. However, the other bridge manager does not mention any selection criteria. With respect to *prioritisation* one bridge manager claims that '*prioritisation has never been necessary because there has been practically no budget constraint',* whereas the other bridge manager defines a detailed list of maintenance measures with descending technical priority from the repair of the wearing coat (first) to clearing the structure from vegetation (last). Furthermore, this bridge manager mentions the high priority of the repair of exposed rebars and of bridges along the abovementioned *VIP-roads* that are very important to traffic and therefore maintained with priority. Regarding the *scheduling* of maintenance measures, both interviewed bridge managers do not describes specific concepts beyond fulfilling technical constraints.

## 3.5 Practical Approaches to Systematic Maintenance Planning

### 3.5.1 Introduction

This section examines by means of interviews how the bridge managers assess and evaluate maintenance strategies and how they decide for their implementation. Specifically, it explores if they apply systematic approaches, such as indicator-based performance assessment and evaluation or sophisticated optimisation procedures. The aim of this investigation is to get an indication of the actual state of the art with respect to systematic maintenance planning and to find out and if there is potential room for improvement by means of new assessment and evaluation techniques.

### 3.5.2 Data Analysis

**Hamburg Bridges**

The interviews reveal that both authorities use the indicator *condition rating*. One authority is using the German software *SIB-Bauwerke* for a standardised condition rating of bridges; the other authority is planning to implement this software in the near future. Further indicators applied by one authority are *total costs* and *costs/m²*; moreover, the same authority plans to develop and introduce an indicator for describing the aspect availability.

Despite the application of these indicators for assessing the actually applied strategies in hindsight, both authorities do not conduct variation studies or similar procedures for assessing alternative strategies as part of a structured maintenance planning process. Only informally, comparison of alternative strategies may be conducted for maintenance planning. Exclusively in the case of the design of new structures, different structural variants are regularly compared by one authority regarding design aspects and the general service life behaviour.

Both authorities also do not apply systematic optimisation procedures for their strategic maintenance planning. Just like the occasional and informal assessment of maintenance strategies, optimisation is left to personal decision of the bridge manager.

**Chennai Bridges**

Both Chennai authorities assess the actually applied maintenance strategies in hindsight with reference to the bridge condition. In doing so, they both use rather vaguely described indicators for a qualitative assessment, such as the *general condition* or the *average condition of the bridge over the last years*. The interviews indicated that this evaluation must be very general because no records are being kept of inspection results and of the applied maintenance measures and also because standardised method for condition rating are not applied.

Besides this qualitative and retrospective assessment of strategies, one authority claims to apply '*long-term cost analyses*' for assessing alternative strategies prior to application. However, maintenance objectives other than costs are not considered for evaluating alternative strategies. The other authority does not conduct any formalised variation studies or similar strategy assessment on a regular base.

Both authorities do without formal optimisation procedures for their strategic maintenance planning. Just like the occasional and informal strategy assessment, also optimisation of maintenance strategies is left to personal decisions of the bridge manager.

## 3.6 Current State of Bridge Management Practice in Hamburg and Chennai

The major finding of the questionnaire and interview survey in Hamburg and Chennai is that the current practice of bridge management can draw benefit from the development of an approach that allows for bridge specific and multi-object-oriented maintenance planning.

In the first place the survey indicates that for both cities the investigated bridges can be clearly differentiated by their *bridge specific profiles of the stakeholders' demands and owner's requirements*, with similar patterns occurring for bridges within similar contexts, such as urban integration, importance to the traffic network, and perception.

A specific preference pattern occurs, for example, in the case of *landmark-type bridges* for which traffic availability and service life becomes the most important aspect, while the aesthetic appearance seems to comparably less decisive . Indeed, these results can be easily explained: Landmark bridges are impressive and mostly attractively designed structures, which usually constitute engineering works of art and therefore are worthy of long-time preservation. On the other hand, they are often situated some distance off densely populated areas because of the big obstacles they cross and also they are mostly carrying main traffic links. Therefore it seems logical that they combine a strong demand for continuously high traffic capacity with lower demand for a pleasant appearance from closer distance.

Another specific pattern can be observed for the *heritage bridges***,** which are worthy of preservation and consequently show a high valuation of the service life in both cities. The importance for traffic availability is also high in both cities because of the strong integration of the bridges into the road system and the high traffic volume in both cities. The only exception to this is the Zollenbrücke in Hamburg, which is exclusively used as a pedestrian bridge with adjacent alternative paths and therefore shows only minor important for traffic. However, the heritage bridges differ between Hamburg and Chennai with respect to the valuation of the aesthetic appearance. The bridges in Hamburg show high ratings of the appearance while the Chennai bridges are rated relatively low. This is mainly due to the criteria applied in the survey, which derive the importance of the appearance mostly from the intensity of visual perception of the bridge. In Chennai the investigated monuments are less integrated into populated areas than in Hamburg; this leads to a lower rating on the appearance scale. This can also be observed at the level of the owner's requirements. Here, the aesthetic aspect is rated a medium '3.0' in Chennai, while the Hamburg monuments are evaluated significantly higher with a mean rating of '4.0'. Likewise, this can be explained by the fact that the investigated monuments in Chennai are less integrated into populated areas.

Also the *city bridges* in Hamburg and Chennai show a relatively balanced valuation of availability and appearance with above-medium ratings together with a comparatively lower rating of service life. The lower rating of the service life can be explained by the fact that the majority of bridges that are part of the local traffic net are rather functional structures. Thus, there is no special necessity of preservation due to socio-cultural reasons. However, at the requirements level the importance shifts from appearance to service life. This change may be explained by a bridge manager's preference for service life, because its consideration together with structural safety and serviceability constitutes the traditional paradigm of bridge management. The above-medium ratings of the other aspects result from the close integration into the traffic system and into populated areas

Bridges of the categories *remote main link* and *remote minor link* have been investigated only in Hamburg. The bridges allocated to these categories perfectly reflect the expected characteristics that follow from the category definition in Table 3.6. Bridges from both categories plausibly show an extremely low importance of the appearance, because as mainly functional structures they are of low cultural interest to the public and, per definition, they are *remotely* located away from populated areas. In terms of traffic importance, the main links shows very high ratings, whereas the minor links are rated well below average. The aspect service life in terms of preservation scores medium values for the main links because their good integration into the environment and—in the case of the Hochstraße Elbmarsch—because of their outstanding technical dimensions. In contrast, the minor links show only a low importance for preservation, because they are mainly functional and structures.

In the second place, the investigation indicates bridge specific strategy differentiation only in Hamburg, while in Chennai one uniform strategy seems to be applied to all bridges. Accordingly, the survey suggests that in Hamburg maintenance planning under consideration of bridge specific preferences is applied. The Chennai authorities on the other hand do not apply different strategies. Consequently, the Chennai authorities should be able to draw benefit from the introduction of multi-objective maintenance planning.

In the third place the investigation shows that neither in Hamburg nor in Chennai structured approaches for maintenance strategy evaluation and assessment are applied. From these findings it can be concluded that the bridge managers both in Hamburg and Chennai can benefit from provision of assessment methods, evaluation techniques and performance indicators which refer to the different preference-aspects.

## 3.7 Conclusion

This chapter describes a survey study of 24 bridges, which has been conducted in cooperation with bridge authorities in Hamburg and Chennai. The investigation with the help of a newly developed questionnaire indicated that bridges can be differentiated by their bridge specific preference profiles in such a way that bridges with similar surroundings, usage, and perception show similar patterns. Thus, the underlying hypothesis of bridge specific requirements is put on firmer ground. However, the investigation also showed that neither in Hamburg nor in Chennai structured approaches for maintenance strategy evaluation and assessment are applied. From this it can be concluded that structured approaches for bridge specific maintenance strategy assessment and evaluation would constitute a stringent improvement to current bridge management practice in both cities.

# 4 A New Approach to Multi-Objective Maintenance Planning

## 4.1 Introduction

From the findings of the chapters 2 and 3 it appears that strategic maintenance planning practice can considerably benefit from an approach that allows for bridge specific and multi-object maintenance planning: The literature review of chapter 2 indicates that the current theoretical concepts have shortcomings because of lacking methods for the development of bridge specific requirements and because of insufficient methods for multi-objective maintenance strategy assessment and evaluation. At the same time, the findings of the survey study in chapter 3 reaffirm the need for considering bridge specific requirements. However, the survey study also shows that maintenance application schemes, when they could be observed in practical application, resulted only from informal consideration of bridge specific peculiarities and not from analytical approaches, which account for these bridge specific requirements.

As a solution to this, a new pragmatic approach to multi-objective maintenance planning is introduced. This approach accounts for the bridge specific context and comprises all steps from the initial requirements/objectives development phase, via strategy assessment, through to multi-objective strategy evaluation.

## 4.2 General Description of the Approach

The new approach to bridge specific and multi-objective maintenance planning comprises the following steps:

- requirements/objectives development phase
- modelling: bridge specific representation of structure, deterioration, and maintenance
- simulation of alternative maintenance strategies
- assessment of the alternative strategies
- evaluation of the alternative strategies
- identification of the most appropriate maintenance strategy—optionally after iterative strategy improvement

The initial requirements/objectives development phase is conducted with the help of the above described questionnaire (sub-section 3.2.4); the items from the second to the fifth block of questions determine the bridge specific stakeholders' demands as well as the owners' requirements and the bridge managers' maintenance objectives. As a result of this phase, the specific preference profile for the bridge is identified and graphically displayed in the form of a star plot diagram by following the procedures that are described in the above sections 3.2.4 and 3.3.

The second to the fourth step—'modelling', 'stochastic simulation', and 'strategy assessment'—are provided by the newly developed SAMS-program, which constitutes the core of the approach (see Fig. 4.1). The SAMS program, whose name is the acronym for *simulation-based assessment of maintenance strategies*, conducts a multi-aspect assessment of maintenance strategies. It repeatedly simulates complete life-cycles—from bridge construction to replacement—for randomly varied input data, which describe the bridge structure, the deterioration mechanisms, and the maintenance effects. On this basis, the SAMS-program calculates distributions of performance indicators for strategy assessment, such as the commonly considered service life and maintenance costs or newly defined indicators that describe traffic related effects or the aesthetic impact.

The evaluation, being the fifth step of the approach, makes use of '*qualitative preference/performance-profile comparison*, which allows for proper multi-objective evaluation of the performance results and which is introduced in sub-section 2.8.5.

As the final step, possibly after iterative strategy improvement and evaluation, the strategy that is most appropriate from the evaluation results can be identified.

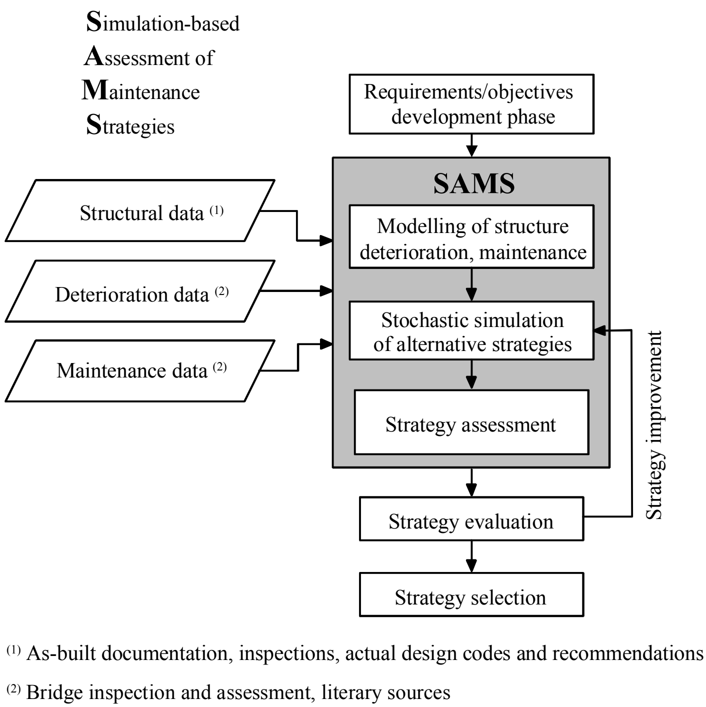



**Fig. 4.1:** The SAMS-program as the core of the new approach for multi-objective maintenance planning

## 4.3 Structural Representation

### 4.3.1 Overview

Within the *SAMS*-program, a bridge structure is represented with respect to three aspects. The first aspect is the load bearing structure itself, and refers to dimensions and material properties. The second aspect is the underlying safety concept and the methods for structural analysis within the SAMS-program. The third aspect comprises the bridge equipment and other components, which do not belong to the actual load bearing structure—for example, the asphalt pavement, the expansion joints, and the visible concrete surfaces.

### 4.3.2 Load Bearing Structure

The load bearing structure is represented by bridge elements which are defined on the basis of cross sections at significant locations. Typical significant locations under bridge specific loading conditions are at the supports, where shear—and in the case of intermediate supports also the negative bending moment—reaches maximum values, and in the span, where the maximum positive bending moment occurs. The structural configuration at these locations is specified in terms of detailed data, for example, the cross sectional area of prestressing steel, concrete strength and geometric dimensions such as the width of the web.

In general, three types of longitudinal bridge elements are chosen. The first type is labelled *pier elements* and represents the negative bending behaviour at a bridge segment where the tendons are located close to the top of the cross section. The second type is labelled *support elements* and represents the locations where the critical shear and torsional loading occurs, which is usually located at a distance *d* (effective depth) from the support (e.g. DIN FB 102). Unlike the above described pier elements, these support elements are localised elements and represent the critical cross section exclusively. The third type is labelled *span elements* and represents the spans, which are primarily subject to positive bending action. Consequently, the representative cross sections of these elements are located at the points of maximum bending moments. The following Fig. 4.2 depicts an example representation of a two-cell box girder bridge by a number of bridge elements and their associated critical cross sections.

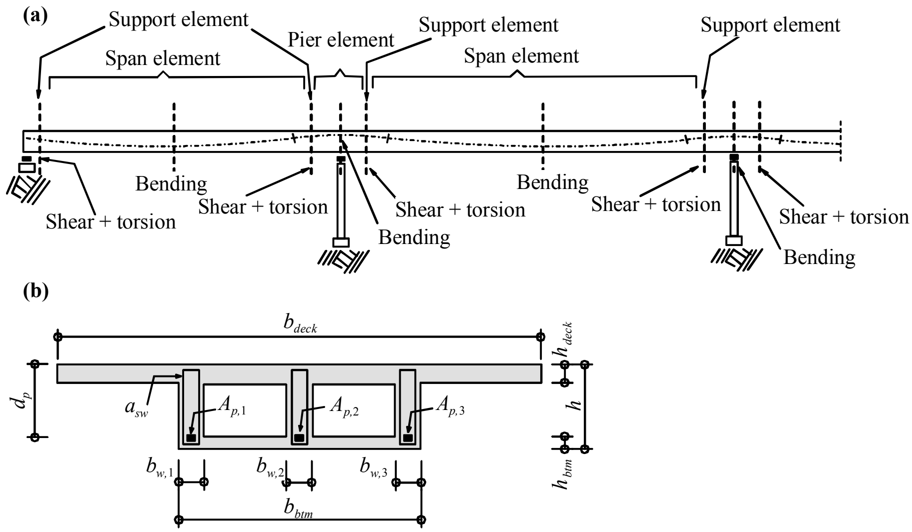



**Fig. 4.2:** Example representation of the load bearing structure within the SAMS program: a) potentially critical locations; b) generic model representation of the cross section at each critical location

In the actual SAMS-layout, which is also used for the case study investigations in the following chapter 5, the load bearing structure is represented only in the longitudinal direction. Also, the program layout is specialised on T-beam and box girder representation because both case study bridges in chapter 5 are prestressed concrete bridges with box girder cross sections. This representation is done in a simplifying way by neglecting the mild steel reinforcement in longitudinal direction because the bounded tendons provide the main contribution to load capacity (see sub-section 4.3.3). Also a simple geometry is assumed in the form of non-inclined webs and in the form of a constant thickness of the deck and bottom slab.

### 4.3.3 Safety Concept and Structural Analysis

#### General Safety Concept

The SAMS program applies the *deterministic code-based approach* for assessing the structural safety, which is described in sub-section 2.7.3. According to this approach the structural safety of the overall structure is expressed in terms of the minimum ratio η of the strength (*R*) to load effect (*S*) that occurs for all significant cross-section and all investigated limit states (see equation 2.6). This is achieved by considering the two limit states ‘bending’ and ‘shear’:

$$\eta_M = \frac{M_{Rd}}{M_{Sd}} \quad ; \quad \eta_{V+T} = \frac{V_{Rd}}{V_{Sd,V+T}} \quad ; \quad \eta = \min\left(\eta_M, \eta_{V+T}\right) \tag{4.1}$$

with

| | |
|---|---|
| $M_{Rd}$ | design value of resistant moment |
| $M_{Sd}$ | design value of the moment, which results from the applied loads |
| $V_{Rd}$ | design value of resistance to shear |
| $V_{Sd,V+T}$ | design value of the shear (shear force and torsion) from the applied loads |

The above design values of the resistances ($M_{Rd}$, $V_{Rd}$) and of the internal forces, which result from the applied loads ($M_{Sd}$, $V_{Sd}$) are calculated following the well known semi-probabilistic *load resistance factor design* (*LRFD*) methodology that is described for example in the DIN FB 102 or the Draft Model Code 2010 (fib 2010a/b). Because maintenance management usually deals with existing bridges, the SAMS-program also considers the guidelines for the re-calculation of existing road bridges of the BMVBS (2011) by applying the complementary rules to the DIN FB 102 of the sophistication level 2, which for example provide less conservative values for the partial safety factors.

In particular, the following basic material parameters and partial safety factors for material properties and actions are applied within the semi-probabilistic LRFD methodology of the DIN FB 102 or the Draft Model Code 2010 or the German *Nachrechnungsrichtlinie* (BMVBS 2011):

**Table 4-1:** Basic material parameters and partial safety factors

| Symbol | Description | Value or formula |
|---|---|---|
| $f_{ck}, f_{yk}, f_{pyk}$ | Characteristic values of concrete strength (*c*), yield stress of mild (*y*) prestressing (*py*) steel | According to relevant codes and standards or inspection results |
| $\gamma_c$, $\gamma_s$, $\gamma_p$ | Partial safety factors for the material properties of concrete (*c*) and mild (*s*) or prestressing (*p*) steel | $\gamma_c = 1.5^{a/b}$<br>$\gamma_s = 1.15^{a/b}$ or $1.05^{c}$<br>$\gamma_p = 1.15^{a/b}$ or $1.10^{c}$ |
| $f_{cd}, f_{yd}, f_{pyd}$ | Design values of concrete strength (*c*) and yield stress of mild (*y*) and prestressing (*py*) steel | $f_{yd/pyd} = \frac{f_{yk/pyk}}{\gamma_s}$ $\quad f_{cd} = \alpha_{cc}\frac{f_{ck}}{\gamma_c}{}^{d}$ |
| $\gamma_G$, $\gamma_Q$, $\gamma_P$ | Partial safety factors for permanent (*G*) and variable actions (*Q*) as well as prestressing (*P*) | $\gamma_G = 1.35^{b/e/f}$ $\quad \gamma_G = 1.20^{c/f/g}$<br>$\gamma_Q = 1.50$<br>$\gamma_P = 1.0$ |
| $\psi$ | Combination factors for simultaneous load occurrence | According to relevant codes and standards |

[a] DIN FB 102 [b] fib 2010a/b [c] BMVBS 2011(level 2) [d] $\alpha_{cc}$=0.85 accounts for adverse long term effects
[e] DIN FB 101 [f] in case of favourable load effects $\gamma$=1.0 (*G*, *P*) or neglection of effects (*Q*)
[g] If detailed investigations of dimensions and specific weights have been conducted at the existing bridge.

**Structural Analysis and Load Effects**

The load effects in the form of internal forces must be externally calculated and be provided to the SAMS program in the form of detailed numerical input data for each of the relevant cross sections. According to the above described semi-probabilistic safety concept, the loads and their combinations are taken from relevant codes and standards. The choice of the method for structural analysis is left to the user of the SAMS program and may comprise simple calculation by hand or sophisticated FE-calculations. The definitions of the relevant loads and the general methods for structural analysis and the calculation of load effects, such as $M_{Sd}$ and $V_{Sd}$ or $T_{Sd}$, are assumed to be well known and thus are not discussed here.

The SAMS-program assesses the structural safety by calculating the values $\eta_M$ and $\eta_{V+T}$ as well as the resulting degree of fulfilment $\eta_{res}$, which is defined as the minimum of $\eta_M$ and $\eta_{V+T}$. The calculation with respect to bending ($\eta_M$) simply requires the design value $M_{Sd}$ as an input value that is compared to the cross sectional resistance $M_{Rd}$, which is calculated internally by the SAMS program as described in the following sub-section. However, the calculation of $\eta_{V+T}$ requires further explanation with respect to the determination of $V_{Sd,V+T,rep}$. This is because the two case study bridges that will be examined in the following chapter 5 are one and two cell box girder bridges, and therefore particular attention is being paid for the determination of $V_{Sd,V+T,rep}$ for these specific cross section types. The term $V_{Sd,V+T,rep}$ describes the resulting shear force in the representative web of the box girder, as it results from both the shear force $V_{Sd}$ and the torsional moment $T_{Sd}$. In the scope of the present thesis, the representative web and the resulting shear force $V_{Sd,V+T,rep}$ are found under the following assumptions:

- The representative location is at distance *d* (effective height) from the support, see for example DIN FB102.
- The shear force $V_{V,g+q}$, which results from dead loads and uniformly distributed live loads, is assumed to be uniformly distributed over the webs (Fig. 4.3 (a)). Thus, each web carries an equal share $V_{V,g+q,web}$ of the total shear force.
- The distribution of the torque $T_{Sd}$ over the webs is assumed to be uniform (Fig. 4.3 (b)).
- At the relevant location at distance *d* from the support, all concentrated variable live loads *Q* are carried by the outer web in the case of a two-cell box girder. In the case of a multi-cell girder, these loads are assumed to distribute over the two adjacent webs according to the respective distance of their application points to the webs (Fig. 4.3 (c)). This is a simplifying assumption of the distribution for loads acting close to the supports; more accurate investigations can be found in Borkowski (2014).
- The distribution of the indirect shear force $V_{P,ind}$ over the webs, which results from secondary prestressing effects, is assumed to be uniform (Fig. 4.3 (d)).

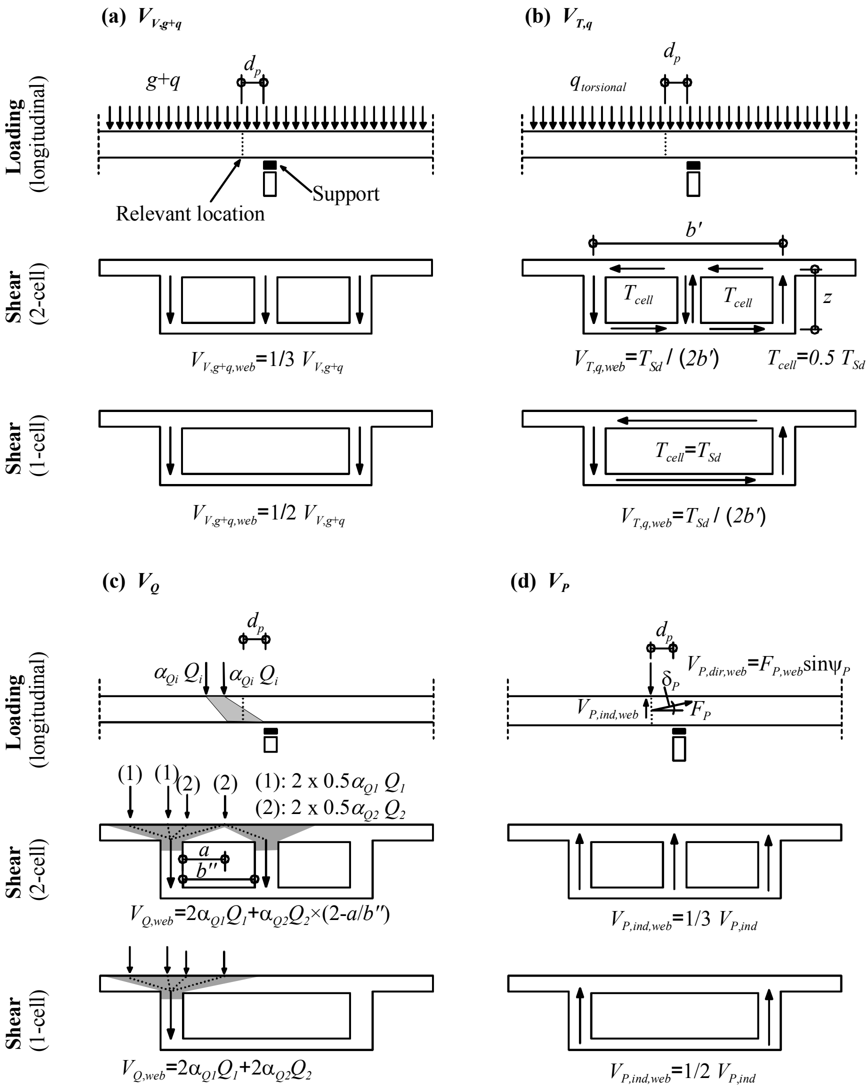



**Fig. 4.3:** Shear forces in the webs of one and two cell box girder cross sections: (a) dead and distributed live loads, (b) from torsional effects of distributed live load, (c) concentrated live load (lorries) close to the support, and (d) prestressing.

Based on these assumptions, the resulting shear force $V_{Sd,V+T,rep}$ for the representative web is calculated. In the example of Fig. 4.3 the representative web is the outer left web where the shear $V_{V,g+q,web}$ and $V_{T,q,web}$ act in the same direction. For the example of the two- and three-cell box girder cross sections in Fig. 4.3 the resulting shear force $V_{Sd,V+T,rep}$ in the outer left web can be calculated under the above described simplifying assumptions by the following equations 4.2 to 4.4. It must be kept in mind that the distributed live load $q$ leads to maximum torsional effects $T_{Sd}$ when it is asymmetrically arranged. This however results in below-maximum shear force effects $V_{V,g+q}$. Consequently, the equations 4.2 to 4.4 must be evaluated for both the arrangements that cause maximum torsional effects and for maximum shear force effects. Only after this the relevant value for $V_{Sd,V+T,rep}$ can be found.

$$V_{Sd,V+T,rep} = V_{V,g+q,web} + V_{T,q,web} + V_{Q,web} + V_{P,web} \tag{4.2}$$

For a two-cell box girder cross section:

$$V_{Sd,V+T,rep} = \frac{V_{V,g+q}}{3} + \frac{T_{Sd}}{2 \cdot b'} + 2\alpha_{Q1}Q_1 + \alpha_{Q2}Q_2 \times \left(2 - \frac{a}{b''}\right) + \frac{V_{P,ind}}{3} - F_{P,web} \sin\psi_P \tag{4.3}$$

For a single-cell box girder cross section:

$$V_{Sd,V+T,rep} = \frac{V_{V,g+q}}{2} + \frac{T_{Sd}}{2 \cdot b'} + 2\alpha_{Q1}Q_1 + 2\alpha_{Q2}Q_2 + \frac{V_{P,ind}}{2} - F_{P,web} \sin\psi_P \tag{4.4}$$

with

| | |
|---|---|
| $\alpha_{Q1}Q_1, \alpha_{Q2}Q_2$ | lorry axle loads (e.g. according to DIN FB 101) |
| $a$ | distance of the concentrated load to the outer left web, see Fig. 4.3 (c) |
| $b'$ | distance between the centriods of the outer webs, see Fig. 4.3 (b) |
| $b''$ | clear width between outer left web and inner web, see Fig. 4.3 (b) |
| $F_{P,web}$ | prestress force in the representative web |
| $\psi_P$ | inclination of the prestressing tendon |
| $V_{P,ind}$ | indirect shear force from prestressing |

**Resistance to Bending ($M_{Rd}$)**

The SAMS program calculates the resistance to bending moments ($M_{Rd}$) for the prestressed box girder cross sections under the following simplifying assumptions:

- Only prestressing steel and optional strengthening by FRP strips or steel plates is regarded.
- The longitudinal mild steel reinforcement is neglected.
- The prestressing steel is assumed to yield at ultimate loading.
- The strengthening material reaches a defined stress level at ultimate.
- All prestressing tendons are represented by a resulting tendon.
- Skew-bending effects, which may arise for example from loading conditions or unsymmetrical reinforcement, are neglected.
- The concrete stresses in the compression zone are represented by a stress block with $\sigma_c = f_{cd}$.
- No longitudinal forces—except for prestressing—are considered.

On the basis of these assumptions, the flexural strength of the simple box girder cross section—such as in the above Fig. 4.2 (b)—is calculated with the help of equivalent T-beam cross sections which are depicted in Fig. 4.4 (a) and (b).

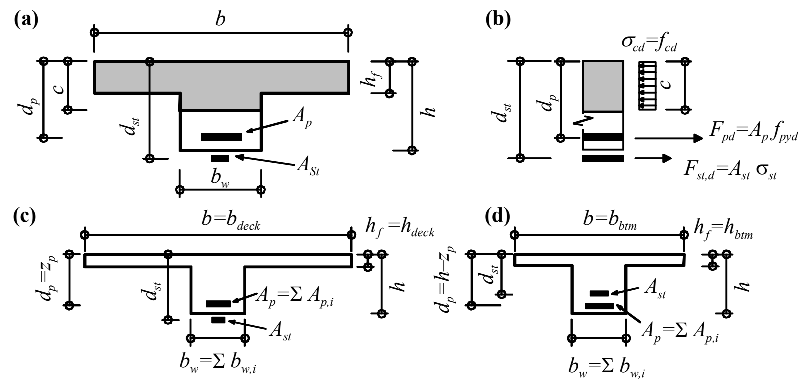



**Fig. 4.4:** Equivalent T-beam cross section for the calculation of $M_{Rd}$: (a) parameters; (b) cross sectional forces and stresses: concrete stresses [$\sigma_{cd}$], prestressing force [$F_{pd}$], force in strengthening material [$F_{st}$]; (c) equivalent cross section where the strengthening material [$A_{st}$] can be applied to the soffit of the bottom slab outside of the cross section; (d) equivalent cross section over the pier where the strengthening material [$A_{st}$] is located at the soffit of the deck slab inside the cross section.

For the equivalent T-beam cross section, the flexural strength is calculated with the help of equation 4.5 if the condition in equation 4.6 applies and thus the compression zone is restricted to the T-beam flange. Alternatively, if equation 4.8 applies and thus the compression zone extends into the web, the resistance to bending is calculated by equation 4.7.

$$M_{Rd} = \frac{c^2}{2}\left(-f_{cd}b\right) + A_p f_{pyd}\, d_p + A_{st}\sigma_{st,d}\, d_{st} \tag{4.5}$$

$$\text{if } c = \frac{A_p f_{pyd} + A_{st}\sigma_{st,d}}{f_{cd} b} \leq h_f \tag{4.6}$$

Or alternatively

$$M_{Rd} = \frac{c^2}{2}\left(-f_{cd}\cdot b_w\right) + A_p \cdot f_{pyd}\cdot d_p + A_{st}\sigma_{st.d}\, d_{st} - f_{cd}\cdot\left(b-b_w\right)\cdot\frac{h_f^2}{2} \tag{4.7}$$

$$\text{and } c = \frac{A_p f_{pyd} + A_{st}\sigma_{st.d} - f_{cd}\cdot h_f\cdot\left(b-b_w\right)}{f_{cd}\cdot b_w} \leq h \tag{4.8}$$

with

| | |
|---|---|
| $c$ | height of the compression zone |
| $b, b_w$ | width of flange, width of web ($w$) |
| $h, h_f$ | total height of the T-beam, height of flange ($f$) |
| $d_p, d_{st}$ | effective depth to prestressed reinforcement ($p$) / to strengthening ($st$) |
| $A_p, A_{st}$ | area of prestressing reinforcement ($p$) / of strengthening component ($st$) |
| $\sigma_{st}$ | stress in strengthening material at ultimate loading of cross section |

**Resistance to Shear ($V_{Rd}$)**

In contrast to the flexural strength $M_{Rd}$, which can be sufficiently described by the above simple mechanical model, the calculation of the shear resistance $V_{Rd}$ is traditionally a disputed topic (Sigrist and Hackbarth 2010). The familiar practical design and assessment rules as well as newly developed models, which are for example based on the *Generalised Stress Field Approach* or the *Cracked Membrane Model*, take into account two important influences on the shear capacity: the stress band inclination (θ) and the effective compressive strength of concrete ($f_{ce}$). However, dispute arises usually about how to treat these two influences—both depend on the cross sectional strain state—in numerical terms in the shear resistance model (e.g. Sigrist 2011; Sigrist and Hackbarth 2010; fib 2010b: 30–38).

The dependency of the stress band inclination θ on the cross sectional strain state is, for example, implicitly taken into account by the German DIN FB 102 for concrete bridges. For the inclination θ it introduces a lower limit, which depends on the cross sectional load effects in terms of the longitudinal force ($N_{Sd}$) and the shear ($V_{Sd}$). The same approach is also followed by the German guideline for the assessment of existing road bridges with a slightly lower minimum value of the inclination θ (BMVBS 2011). Also, both approaches consider the influence of the strain state on the compressive strength in terms of a constant reduction factor $\alpha_c$. In contrast, the theoretical model that has been proposed by Sigrist (2011) applies explicit relationships between the principal strain on the one side and the compressive strength as well as the stress band inclination on the other side.

The SAMS-program provides two alternative methods for the calculation of the shear capacity, which are based on two of the above mentioned approaches—the Generalised Stress Field Approach (GSFA) and the method according to the DIN FB 102 and BMVBS (2011).

The first method is based on the Generalised Stress Field Approach (GSFA) and applies an iterative procedure, which is introduced in detail by Sigrist (2011), Hackbarth and Sigrist (2011) and Sigrist and Hackbarth (2010). This method involves the following steps, which will be described in detail below:

- estimation of the stress band inclination θ
- calculation of the strain mid-depth ($\varepsilon_x$) and the principal strain ($\varepsilon_1$) in the web
- calculation of the effective compressive strength ($f_{ce}$)
- calculation of the shear resistance of the reinforcement ($V_{Rd,s}$) and the concrete ($V_{Rd,c}$)
- iteration with altered inclination θ until $V_{Rd,s}= V_{Rd,c}$

As a first step, the stress band inclination θ must be estimated because all the following parameters depend on this parameter. The starting value for θ is always chosen as 20°. This minimum value is chosen according to Sigrist (2010), who proposes the following limits for θ in the case of reinforcing steel with $f_{yk}$= 500 MPa and regular yield characteristics:

$$20° + 5000\varepsilon_x \leq \theta \leq 35° + 5000\varepsilon_x \tag{4.9}$$

The strain $\varepsilon_x$ in the mid-depth of the girder is calculated on the basis of the simple calculation that is presented in Marti (1994) and is also applied by Hackbarth and Sigrist (2011). This approach is based on stress field analysis and—on the safe side—sets the strain in the compression chord to zero. The following Fig. 4.5 depicts the basic assumptions for the calculation of the strain $\varepsilon_x$:

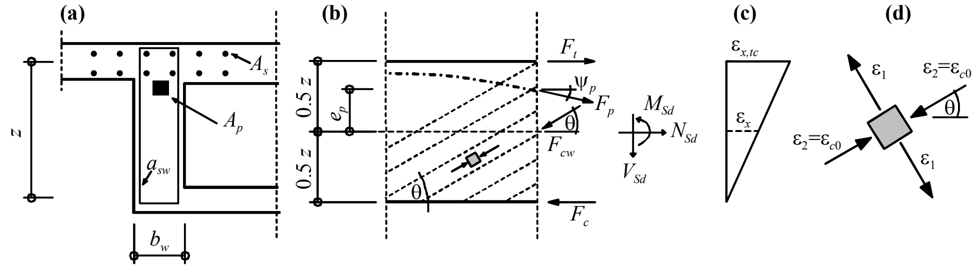



**Fig. 4.5:** Estimation of the strain $\varepsilon_x$ in the mid-depth of a box girder: (a) Web with mild steel in the tension chord ($A_s$), prestressing tendons ($A_p$), and shear reinforcement in the form of non-inclined stirrups ($a_{sw}$); (b) internal forces and sectional forces and moments; (c) strain $\varepsilon_x$ in the mid-depth and overall distribution over the web; (d) principal strains in the stress band. (Note: The figures (b) and (c) are based on Fig. 4 in Hackbarth and Sigrist (2011)).

This simple approach calculates the strain $\varepsilon_x$ in mid-depth from the simple linear distribution (Fig. 4.5 (c)) as half the maximum strain that occurs in the tension chord ($\varepsilon_{x,tc}$). The strain $\varepsilon_{x,tc}$ results from the overall tension force $F_t$ and it also depends on the amount of mild and prestressing steel in the tension chord. Simplifying it is assumed, that 50% of the prestressing tendons are located in the tension chord. Also the longitudinal force $N_{Sd}$ (which does not include direct prestressing effects) is assumed to be zero. Under these assumptions, the overall tension force $F_t$ and the strain $\varepsilon_x$ result from the following equations:

$$F_t = -\frac{M_{Sd}}{z} + \frac{V_{Sd}}{2}\cot\theta - \frac{F_p}{2}\cos\psi_p\left(\frac{2e_p}{z}+1\right) \tag{4.10}$$

$$\varepsilon_x = \frac{F_t}{2\left(E_s A_s + 0.5 E_p A_p\right)} \tag{4.11}$$

with

| | |
|---|---|
| $M_{Sd}, V_{Sd}$ | internal forces from structural analysis |
| $F_p$ | prestressing force |
| $z$ | effective shear depth |
| $e_p$ | distance of tendon from the central axis |
| $\theta, \psi_p$ | inclination of stressband ($\theta$) and tendon ($\psi_p$) |
| $E_s A_s, E_p A_p$ | tension stiffness of mild and prestressing steel |

The principal tensional strain $\varepsilon_1$ that occurs transversally to the compressive strain $\varepsilon_2$ (see Fig. 4.5 (d)) is calculated by setting $\varepsilon_2 = \varepsilon_{c0} = -2‰$ which is the concrete strain at peak stress in compression:

$$\varepsilon_1 = \varepsilon_x + (\varepsilon_x - \varepsilon_2)\cot^2\theta \tag{4.12}$$

Then, the effective compressive strength of concrete $f_{ce}$ is calculated with the help of a reduction factor $\eta_c$ that depends on the principal strain:

$$\eta_c = \frac{1}{1.2 + 55\varepsilon_1} \tag{4.13}$$

$$f_{ce,d} = \eta_c \frac{f_{ck}}{\gamma_c}\left(\frac{30}{f_{ck}}\right)^{1/3} \tag{4.14}$$

The shear resistance of the concrete $V_{Rd,c}$ results directly from the effective compressive strength. This resistance is compared to the shear resistance of the reinforcement $V_{Rd,s}$ in order to validate the initially chosen stress band inclination $\theta$. Here, the term $V_{Rd,s}$ also includes the effect of possibly applied strengthening.

$$V_{Rd,c} = f_{ce,d}\, b_w\, z \sin\theta\cos\theta \tag{4.15}$$

$$V_{Rd,s} = \left(a_{sw} f_{yd} + a_{st}\,\sigma_{st}\right) z \cot\theta \tag{4.16}$$

with

| | |
|---|---|
| $b_w$ | width of web |
| $a_{sw}$ | transverese shear reinforcement $[cm^2/m]$ |
| $a_{st}$ | transverese shear strengthening $[cm^2/m]$ |
| $\sigma_{st}$ | stress in strengthening material at ultimate loading of cross section |

If both shear resistances are found to be equal, the angle $\theta$ that leads to the maximum shear capacity is found. This is because the resulting shear resistance is defined as the minimum of $V_{Rd,s}$ and $V_{Rd,c}$. and with increasing $\theta$ the value of $V_{Rd,s}$ decreases while the value of $V_{Rd,c}$ increases (Fig. 4.6 (a)). Otherwise the above described steps must be repeated with altered values for θ until $V_{Rd,s}$ becomes equal to $V_{Rd,c}$.

The following Fig. 4.6 (b) illustrates that this iterative procedure is equivalent to finding the zero of the function $f = V_{Rd,s} - V_{Rd,c}$, which can be explicitly expressed by substituting the above equations 4.10 to 4.14 into the expressions for $V_{Rd,s}$ and $V_{Rd,c}$ (equations 4.15 and 4.16). In SAMS this is implemented by applying a simple iterative secant method for finding the zero of the function $f(\theta)$.

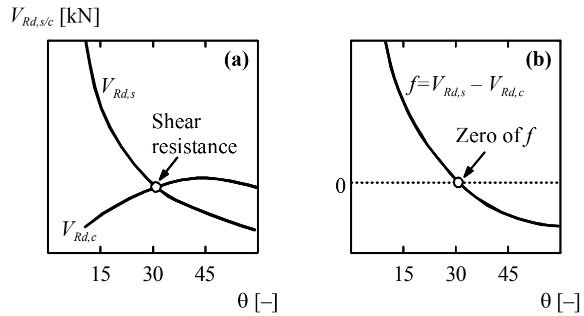



**Fig. 4.6:** (a) $V_{Rd,s}$ and $V_{Rd,c}$ as functions of the stress band inclination θ and resulting shear resistance; (b) function $f=V_{Rd,s}-V_{Rd,c}$.

However, this GSFA-based iterative calculation is time consuming—especially if an extensive number of shear capacity calculations is necessary, which is the case for strategy assessment over a long time horizon and many potentially critical bridge elements. In such a case an alternative methodology can be applied by the SAMS program. This methodology is based on the regulations of the DIN FB 102 and the less conservative lower minimum value of the inclination θ as it is demanded by the German guideline for the re-calculation of road bridges (BMVBS 2011). This method allows for a direct calculation of the shear capacity without iterative calculations. It uses equations for $V_{Rd,c}$ and $V_{Rd,s}$ analogous to the Generalised Stress Field Approach (GSFA), but differs in two aspects.

First, the effective compressive strength of concrete $f_{ce,d}$, which is accounted for by the factor $\eta_c$ in the case of the GSFA, is calculated by a constant reduction factor $\alpha_c$=0.75:

$$f_{ce,d} = \alpha_c f_{cd} = 0.75 f_{cd} \tag{4.17}$$

Second, the limits for θ are calculated by the following equation 4.18. This equation describes the empirical connection between θ on the one side and the longitudinal stresses $\sigma_{cd}$ as well as the tensional strength of concrete $f_{ct}$, which is proportional to $(f_{ck})^{1/3}$, on the other side (e.g. Rombach 2010: 391).

$$0.58 \leq \cot\theta \leq \frac{\left(1.2 - 1.4 \times \sigma_{cd} / f_{cd}\right)}{\left(1 - V_{Rd,c} / V_{Sd}\right)} \leq 2.50 \tag{4.18}$$

with

$$\sigma_{cd} = N_{Ed} / A_c \quad \left(\text{including } F_p\text{, negative in case of compression}\right)$$

$$V_{Rd,c} = \left[0.24 \times f_{ck}^{1/3} \times \left(1 + 1.2 \times \sigma_{cd} / f_{cd}\right)\right] \times b_w \times z$$

Third, the inclination $\theta_{maxR}$ that leads to the maximum shear capacity is found by simply solving the equation $V_{Rd,s} = V_{Rd,c}$ for θ. It can be easily seen from Fig. 4.6 that only inclinations ≤ 45° are relevant for calculating the maximum bearable shear because $V_{Rd,c}$ reaches its peak at that angle. Following from these considerations, the inclination $\theta_{maxR}$ can be found under consideration of the limiting values according to equation 4.18 by the following equation 4.19:

$$1 \leq \cot\theta_{\max R} = \sqrt{\frac{\alpha_c \cdot f_{cd} \cdot b_w}{a_{sw}\, f_{yd} + a_{st}\, \sigma_{st}} - 1} \leq \frac{\left(1.2 - 1.4 \times \sigma_{cd} / f_{cd}\right)}{\left(1 - V_{Rd,c} / V_{Sd}\right)} \leq 2.50 \tag{4.19}$$

In the case that $\theta_{maxR}$ is smaller than 45°, the resistance to shear is found by setting $\theta = \theta_{maxR}$ in the above equation 4.16, which expresses the shear capacity $V_{Rd,s}$ of the transversal reinforcement. Otherwise, the resistance to shear must be calculated by setting θ=45° in equation 4.15, which expresses the shear capacity $V_{Rd,c}$ of the concrete strut. This is because $V_{Rd,c}$ reaches its maximum value at θ=45°.

The advantage of the method according to the DIN FB 102 is that it does not require iterative calculations and thus is more economic in terms of calculation time. An example calculation of the shear resistance, which is based on the example bridge from Hackbarth and Sigrist (2011), revealed that the iterative GSFA method requires a 2.5-fold calculation time compared to the approach of the DIN FB 102 and BMVBS (2011). On the other hand, the advantage of the GSFA method is that it provides a structured approach whose results show good correlation with experimental data (Sigrist 2010).

However, with respect to the conservativeness of both approaches, no general statement can be made (Sigrist and Hackbarth 2010: 692–693). An example for this is given by the following Fig. 4.7, which compares the shear resistance of the example bridge from Hackbarth and Sigrist (2011) for a varying amount of shear reinforcement for two alternative values of the compressive strength. As can be seen in this figure, the DIN-approach is more conservative for a compressive strength of 30 MPa, while the GSFA-approach shows comparatively more conservative results at high reinforcement ratios in the case of a higher compressive strength (40 MPa).

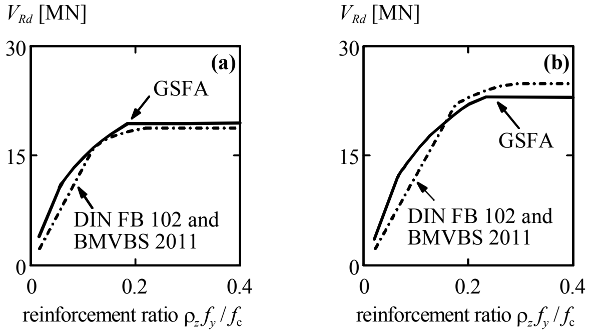



**Fig. 4.7:** $V_{Rd}$ for two alternative values for $f_{ck}$: (a) $f_{ck}$=30MPa and (b) $f_{ck}$=40MPa.

For this reason, the SAMS-program provides both approaches so that the user can choose his preferred method. Usually, when large numbers of calculations become necessary for strategy assessment, the DIN FB 102 approach will be the most convenient, while the GSFA may be chosen as the alternative method when the bridge shows to be critical in shear so that a more refined method should be applied. An in-depth study of the GSFA-application to reinforced and prestressed concrete beams in shear has been conducted by Hackbarth (2014).

### 4.3.4 Bridge Equipment and Surfaces

#### Representation

The bridge equipment and the visible surfaces decisively influence the performance of the bridge. Above all, their condition and individual performance directly determine how the bridge interacts with the demands of the stakeholders: The condition of the surfaces mainly affects the aesthetic performance of the bridge—and possibly the traffic safety of an underpassing traffic route—whereas the bridge equipment condition can affect the traffic flow on the bridge. In addition, the performance of the equipment and the surfaces in many cases directly or indirectly influences the structural safety of the overall bridge. An example for direct influence is found in the form of deteriorated bearings and expansion joints, which can affect the structural safety if they, due to their deteriorated condition, restrict movements they have been designed for. An example for indirect influence can be found in leaking expansion joints, which can have a significant effect on the deterioration of the adjacent structural elements. Similarly, the condition of the asphalt pavement, of the drainage system, and of the surfaces has a strong influence on deterioration of the load bearing structure and thus it indirectly influences the structural safety.

Within the SAMS program, all the bridge equipment parts and the surfaces are specified by numerical condition indices (*CI*) of the

- concrete surfaces of webs, cantilever soffits, bottom slab soffit and kerbs/sidewalk ($CI_{CS,web}$, $CI_{CS,cant}$, $CI_{CS,btm}$, $CI_{CS,kerb}$)
- bearings ($CI_{Bear}$)
- expansion joints ($CI_{Exp}$)
- asphalt pavement ($CI_A$)
- drainage system ($CI_{Drain}$)

The equipment components and surfaces are generally distributed along the bridge, such as bearings, drain pipes and drain inlets. Within the SAMS-program, the equipment and surfaces are allocated to the bridge elements that are described in the above sub-section 4.3.2 and which are representing the load bearing structure. Hereby influences of the equipment and surface condition on the development of the structural condition can be better accounted for—especially concurrent maintenance effects on both the equipment and the load bearing structure can be plausibly defined. This allocation to the bridge elements is achieved by means of weighting factors $w_i$, which for example can be derived according to the below rules:

- Localised bridge equipment, such as bearings and expansion joints, is allocated to the nearest representative cross section of the bridge. The relevant weighting $w_i$ is then defined as the ratio of the number of allocated components to the total number of components. (see equation 4.20)
- Distributed components, such as the drainage system or the asphalt pavement, are allocated to one or more of the span elements. The relevant weighting $w_i$ is then defined as the ratio of the element's representative length to the overall bridge length. (see equation 4.21)

$$w_{Bear,i} = \frac{N_{Bear,i}}{N_{Bear}} \quad \text{and} \quad w_{Exp,i} = \frac{N_{Exp,i}}{N_{Exp}} \tag{4.20}$$

$$w_{Drain,i} = w_{AP,i} = w_{CS,component,i} = \frac{l_{Element,i}}{l_{Bridge}} \tag{4.21}$$

with

| | |
|---|---|
| $N_{Bear/Exp}$ | total number of bearings/expansion joints |
| $N_{Bear/Exp,i}$ | number of bearings/expansion joints allocated to element 'i' |
| $l_{Element,i}$ | representative length of the i$^{th}$ element |
| $l_{Bridge}$ | total length of bridge |

**Element-level Condition Index**

As described in sub-section 2.7.2 of this thesis, there are two basic methods for calculating a condition index: *worst single value rating* and *weighted superposition*, which both are applied by the SAMS program. A detailed description regarding the actual implementation of these alternative calculation methods can be found in sub-section 4.7.2 where the performance indicators for bridge condition are described and discussed in terms of the underlying approaches.

As for the time being, suffice it to say that only the concrete surface condition index is calculation from components below the SAMS-element level: The concrete surface condition index $CI_{CS,i}$ describes the resulting condition of the concrete surface at the element-level by condensing the condition of the web, the bottom slab, the cantilevers, and the kerbs into one single index. It is calculated by means of weighted superposition according to equation 4.22.

$$CI_{CS,i} = w_{web,i} \times CI_{CS,web,i} + w_{cant,i} \times CI_{CS,cant,i} + w_{btm,i} \times CI_{CS,btm,i} + w_{kerb,i} \times CI_{kerb,i} \quad (4.22)$$

with

$$w_{web} + w_{cant} + w_{btm} + w_{kerb} = 1.0$$

The above weighting factors, such as $w_{web,i}$, for the calculation of $CI_{CS,i}$ represent the relative importance of the webs, the bottom slab soffit, the kerbs/sidewalks, and the cantilevers for the overall impression of the bridge and should be chosen accordingly. For example a low rise motorway underbridge may be given a comparatively low $w_{btm}$ value and a higher $w_{web}$ value, in contrast to a bridge in the city with pedestrian traffic underneath, which should be given more balanced values for the $w_{btm}$ and $w_{web}$ weighting factors. All other equipment components are represented by single condition indices, such as $CI_A$ and $CI_{Exp}$, and therefore do not need a superposition on the element level.

The SAMS algorithm uses a rating scale for the condition indices in the range from 1 ('*excellent*') to 0 ('*very poor*') in order to allow for an intuitive interpretation in analogy to a percentage value. However, this is in contrast to common practical applications, as for example the rating scale of the German RI-EBW-PRÜF (2007, 2013), which ranges from 0 (best) to 4 (worst) for individual elements and from 1 to 4 in the case of the overall structure, or the rating scale that is applied for New York City bridges by Dubin and Yanev (2001: 6) and is ranging from 7 (best) to 1 (worst). Nevertheless, if preferred by the user of the SAMS program, the resulting ratings can be easily transformed to alternative scales. For instance, the falling 1–0 scale ($CI_{SAMS}$) can be transformed by the following linear relationship to the rising 0–4 and 1–4 scales of the German RI-EBW-PRÜF ($CI_{RI\text{-}EBW\text{-}PRÜF}$). These two scales refer to individual elements (0–4) and to entire bridge structures (1–4), respectively.

$$CI_{RI-EBW-PRÜF,individual\ element} = 4 - 4CI_{SAMS,individual\ element} \quad (4.23)$$

or

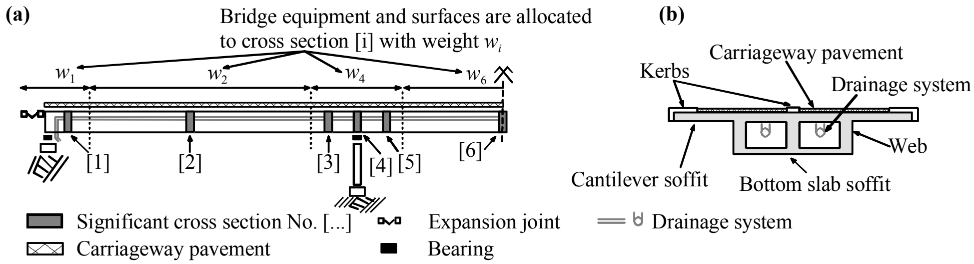
$$CI_{RI-EBW-PRÜF,bridge} = 4 - 3CI_{SAMS,bridge}$$

### 4.3.5 Overall Structural Representation of the Bridge

As described above, the bridge structure is represented within the SAMS program by bridge elements. These elements on the one hand represent the load carrying structure in the form of significant cross sections. On the other hand, these elements represent the bridge equipment and surfaces in the form of condition indices and weighting factors. In general, the data that represent the bridge structure refer to four different facets:

- geometrical dimensions (e.g. height, width, area)
- material properties (e.g. strength)
- internal forces (e.g. $M_{Sd}$)
- condition of the bridge equipment and surfaces (*CI*) and allocation parameters (*w*)

An example representation is depicted in Fig. 4.8, which exemplifies the structural representation of a symmetrical three span prestressed concrete box girder bridge. Based on simple structural considerations, the bridge is represented by six bridge elements ([1] to [6]), which include significant cross sections with high bending and shear actions (Fig. 4.8 (a)). These six bridge elements are contained in the six-dimensional data array *Sys_CrossSections*(), as is depicted in Fig. 4.8 (c). Each of the elements (Fig. 4.8 (d)) contains detailed data regarding dimensions and material properties as well as regarding the local load actions in the form of the prestressing force $F_p$, the bending moment $M_{Sd}$ and the resulting shear force $V_{Sd,V+T,res}$ for the representative web. Furthermore, the bridge equipment as well as the visible concrete surfaces (Fig. 4.8 (b)) are accounted for. This is done by allocating the localised equipment—expansion joints and bearings—as well as the longitudinally distributed concrete surfaces, the pavement, and the drainage system by means of weighting factors $w_i$ to the bridge elements [1] to [6].




**(c)** Overall representation by bridge elements referring to cross section [i]

$$Sys\_CrossSections = \begin{bmatrix} Sys\_CrossSections(1) \\ Sys\_CrossSections(2) \\ Sys\_CrossSections(3) \\ Sys\_CrossSections(4) \\ Sys\_CrossSections(5) \\ Sys\_CrossSections(6) \end{bmatrix} = \begin{bmatrix} \text{cross section [1]: western abutment} \\ \text{cross section [2]: first span} \\ \text{cross section [3]: left of first pier} \\ \text{cross section [4]: first pier} \\ \text{cross section [5]: right of first pier} \\ \text{cross section [6]: second span} \end{bmatrix}$$

**(d)** Detailed data of the bridge elements

$$Sys\_CrossSections(i) = \begin{bmatrix} \left.\begin{matrix} Sys\_CrossSections(i).b_{deck} \\ \ldots \\ Sys\_CrossSections(i).\delta_{st} \end{matrix}\right\} \text{Dimensions} \\ \left.\ldots\right\} \text{Material properties} \\ \left.\ldots\right\} \text{Internal forces} \\ \left.\ldots\right\} \text{Equipment and surfaces} \end{bmatrix}$$

with:

**Dimensions**
$Sys\_CrossSections(i).b_{deck}$
$Sys\_CrossSections(i).h_{deck}$
$Sys\_CrossSections(i).b_{btm}$
$Sys\_CrossSections(i).h_{btm}$
$Sys\_CrossSections(i).h$
$Sys\_CrossSections(i).A_p$
$Sys\_CrossSections(i).d_p$
$Sys\_CrossSections(i).A_{st}$
$Sys\_CrossSections(i).d_{st}$
$Sys\_CrossSections(i).a_{sw}$
$Sys\_CrossSections(i).a_{st}$
$Sys\_CrossSections(i).b_{w,res}$
$Sys\_CrossSections(i).b_{w,crit}$
$Sys\_CrossSections(i).z$
$Sys\_CrossSections(i).\delta_p$
$Sys\_CrossSections(i).\delta_{asw}$
$Sys\_CrossSections(i).\delta_{st}$

**Material properties**
$Sys\_CrossSections(i).f_{pyk}$
$Sys\_CrossSections(i).E_p$
$Sys\_CrossSections(i).f_{yk}$
$Sys\_CrossSections(i).E_s$
$Sys\_CrossSections(i).f_{ck,web}$
$Sys\_CrossSections(i).f_{ck,btm}$
$Sys\_CrossSections(i).f_{ck,deck}$
$Sys\_CrossSections(i).\sigma_{st}$
$Sys\_CrossSections(i).\delta_{ck,web}$
$Sys\_CrossSections(i).\delta_{ck,btm}$
$Sys\_CrossSections(i).\delta_{ck,deck}$

**Internal forces**
$Sys\_Demand(i).M_{sd}$
$Sys\_Demand(i).F_p$
$Sys\_Demand(i).V_{Sd,V+T,res}$

**Equipment and surfaces**
$Sys\_CrossSections(i).CI_A$
$Sys\_CrossSections(i).w_A$
$Sys\_CrossSections(i).CI_{CS,web}$
$Sys\_CrossSections(i).CI_{CS,cant}$
$Sys\_CrossSections(i).CI_{CS,btm}$
$Sys\_CrossSections(i).CI_{CS,kerb}$
$Sys\_CrossSections(i).w_{CS}$
$Sys\_CrossSections(i).CI_{Exp}$
$Sys\_CrossSections(i).w_{Exp}$
$Sys\_CrossSections(i).CI_{Bear}$
$Sys\_CrossSections(i).w_{Bear}$
$Sys\_CrossSections(i).CI_{Drain}$
$Sys\_CrossSections(i).w_{Drain}$

**Fig. 4.8:** Example SAMS representation: (a) Elevation of example bridge with six elements representing the significant cross sections [1] to [6] as well as the bridge equipment and surfaces; (b) cross section with marking of visible concrete surfaces; (c) and (d) data organisation within the SAMS Matlab-code.

## 4.4 Deterioration Mechanisms

### 4.4.1 General Implementation of Deterioration Mechanisms in SAMS

The deterioration mechanisms describe the adverse development of structural parameters, such as the area of prestressing steel ($A_p$), over a period of time as a result of mechanical, physical or chemical actions. The present thesis considers nine deterioration mechanisms, ranging from corrosion of prestressing and mild steel to the deterioration of the drainage system:

- corrosion of prestressing steel
- corrosion of mild steel
- deterioration of FRP strengthening
- reduction of compressive strength of concrete
- deterioration of the concrete surface
- deterioration of the asphalt pavement
- deterioration of the expansion joints
- deterioration of the bearings
- deterioration of the drainage system

All deterioration mechanisms are assumed to follow a multi-linear course as depicted in Fig. 4.9 (a) and are described in the form of equation 4.24. The deterioration courses are described by means of the reduction factor $\delta(t)$ which is defined as the ratio of the initial value (e.g. strength) to the value at the time $t$ (Dette and Sigrist 2009). The deterioration initiates after a period $T_i$ with a rate $\alpha_1$ until a level $\delta_{k1}$ is reached. Then, the deterioration proceeds with rate $\alpha_2$ until a final rate $\alpha_3$ occurs at level $\delta_{k2}$. By this approach, various courses such as constant deterioration ($\alpha_1 = \alpha_2 = \alpha_3$) or a diminishing rate ($\alpha_1 > \alpha_2 > \alpha_3$) can be accounted for.

$$\delta\left(t, T_{kj}\right) = \delta_{kj} - \alpha_{j+1}\left(t - T_{kj}\right) \quad \text{for } t > T_i \quad \text{and} \quad \delta = 1.0 \text{ for } t \le T_i \tag{4.24}$$

with

$\delta$ — deterioration level of a structural parameter: ratio of actual value to the initial value, e.g. $\delta_c = f_{ck}(t) / f_{ck}(t=0)$

$\alpha_j$ — $\alpha_0 = 0 \ (t \le T_i)$, $\alpha_1 (1 \le \delta \le \delta_{k1})$, $\alpha_2 (\delta_{k1} < \delta \le \delta_{k2})$, $\alpha_3 (\delta_{k2} < \delta)$

$\delta_{kj}$ — deterioration level that marks a change in the rate $\alpha$ $(\delta_{k0} = 1)$

$T_{kj}$ — time, at which the deterioration level $\delta_{kj}$ is reached $(T_{k0} = T_i)$

In the scope of this thesis, a quadrilinear deterioration curve is regarded to be sufficient in order to account for the main characteristics of a deterioration process: initiation and several changes of deterioration rate with potential lower limits for the level of deterioration by setting $\alpha$=0 (Fig. 4.9). Such a lower limit, for example, can occur if a maximum decrease of concrete strength due to alkali-aggregate reaction is considered (Fig. 4.9 (b)).

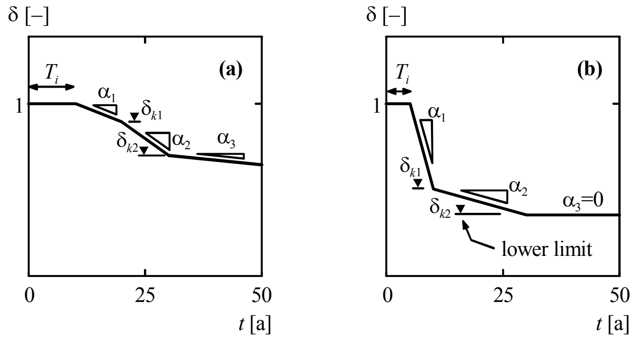



**Fig. 4.9:** Basic deterioration mechanisms: (a) General development over an example period of 50 years; (b) special case with deterioration limit at $\delta_{k2}$.

It is quite clear that the deterioration intensity, which is expressed by the values for the parameters ($T_i$, $\alpha_1$, $\alpha_2$, $\alpha_3$), depends on local particularities regarding the structural layout as well as the pollutant load, especially in terms of chlorides and water. A typical example for this is pronounced corrosion in the vicinity of expansion joints (e.g. Dette 2008). Also concrete strength reduction can differ according to the location within the cross section (web, deck, and bottom). For instance, the deck slab may be subject to high alkali-silica reaction because of leaks in the asphalt pavement, while the bottom slab may be spared from water and chloride attack due to a sound drainage system. An analogous differentiation can also be made for the condition of the concrete surface ($CI_{CS,web}$, $CI_{CS,cant}$, $CI_{CS,btm}$, $CI_{CS,kerb}$).

In order to account for these circumstances, the SAMS program distinguishes between three deterioration intensity levels for low (L), medium (M) and high (H) vulnerability and pollutant load. These intensity levels are defined in terms of value distributions for the deterioration parameters. They are specified by an upper and lower bound as well as by a central value of the respective distribution. Simplifying, the present SAMS version uses value ranges only for the parameters $T_i$ and $\alpha_1$, while the $\alpha_2$, $\alpha_3$ values are directly derived from the values of $T_i$ and $\alpha_1$.

The SAMS-program offers the choice between two alternative value distributions: the beta-distribution and the triangle-distribution. These two types have been favoured over the Normal distribution or other distribution types, including the lognormal distribution, because they both are clearly limited to defined value ranges and thus anomalies like negative or extremely high values for $\alpha_1$ or $T_i$ are avoided. Alternatively, the SAMS program also allows for a deterministic selection of deterioration parameters. Also percentiles of the $\alpha_1$ and $T_i$ distributions can be calculated by the SAMS program and be used as deterioration parameters.

The following figures Fig. 4.10 and Fig. 4.11 exemplify triangular distributions and beta-distributions for the deterioration parameters $T_i$ and $\alpha_1$ and the resulting deterioration courses for a stochastic sample for each intensity level. The deterioration courses are calculated according to equation 4.24 by $\delta=1-\alpha(t-T_i)$ under the assumption $\alpha=\alpha_1=\alpha_2=\alpha_3$. The value ranges of the distributions can be freely defined—they can be adjacent (Fig. 4.10) or they can overlap (Fig. 4.11); there may also be a gap between the value ranges of the different deterioration intensities.

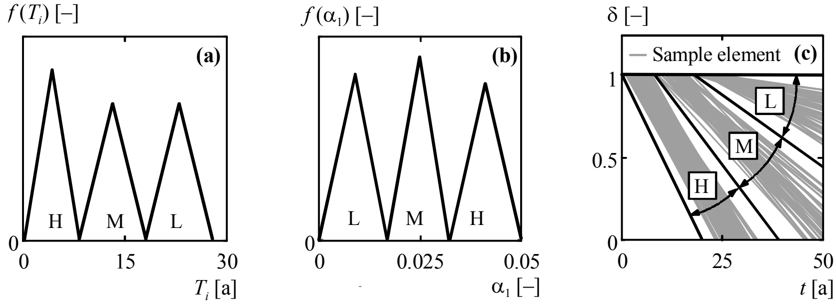



**Fig. 4.10:** Example triangular distributions of the $T_i$ (a) and the $\alpha_1$ (b) parameter values for the deterioration intensity levels L, M, H and the respective example deterioration courses (c). (*Note*: The bold curves mark the limits of each intensity level; the sample size is $N$=50).

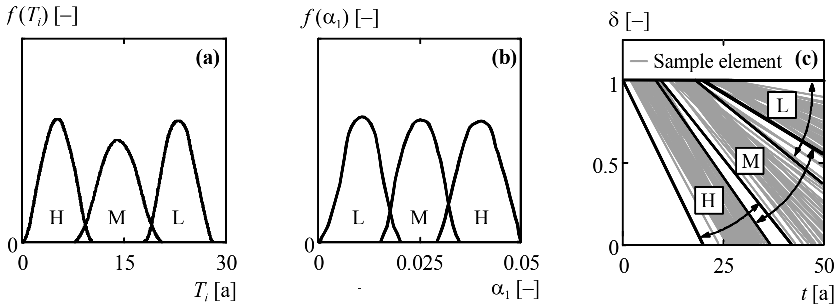



**Fig. 4.11:** As in the above Fig. 4.10, with overlapping value ranges and beta-distributions of $T_i$ and $\alpha_1$ (sample size $N$=50).

Accounting for local particularities, the present thesis considers nine deterioration mechanisms, which are listed in Table 4-2 with local differentiation. By this differentiation, for example, the deterioration curve for the deck slab concrete ($f_{ck,deck}$) may be specified by high-intensity (H) deterioration parameters, while the bottom slab concrete strength deterioration parameters may be specified by low-intensity (L) deterioration. Also—as deterioration intensities may also vary along the bridge—the bottom slab deterioration can be defined as medium or even intensity at another SAMS element, which for example could be the one that represents the characteristic conditions at a leaking expansion joint.

**Table 4-2:** Deterioration mechanisms, local differentiation, and effects

| Deterioration mechanism | Local differentiation | Effect on... [a] |
|---|---|---|
| Prestressing steel corrosion | Corrosion of prestressing steel | $A_p$ |
| Mild steel corrosion | Corrosion of shear reinforcement | $a_{sw}$ |
| Deterioration of FRP-strengthening | Deteriorating bending strengthening | $A_{st}$ |
| | ...shear strengthening | $a_{st}$ |
| Concrete strength reduction | Strength reduction in the web | $f_{ck,web}$ |
| | ... in the bottom slab | $f_{ck,btm}$ |
| | ... in the deck slab | $f_{ck,deck}$ |
| Deterioration of concrete surface | Deterioration of web surface | $CI_{CS,web}$ |
| | ... cantilever surface | $CI_{CS,cant}$ |
| | ... bottom slab surface | $CI_{CS,btm}$ |
| | ... kerb surface | $CI_{CS,kerb}$ |
| Asphalt pavement deterioration | | $CI_A$ |
| Expansion joint deterioration | | $CI_{Exp}$ |
| Bearing deterioration | | $CI_{Bear}$ |
| Drainage system deterioration | | $CI_{Drain}$ |

[a] See for example Fig. 4.8 (d) or *Symbols and Notation*

The following sub sections portray the considered deterioration mechanisms and outline the characteristic influences with respect to the deterioration intensities.

### 4.4.2 Corrosion of Steel (Mild Steel and Prestressing Steel)

#### Initiation Time ($T_i$)

The relevant literature (e.g. Tuuti 1982, Böhni 2005) explains that corrosion of steel in concrete requires the simultaneous fulfilment of four conditions in order to take place as an electrochemical process. First, oxygen and water and, as a second precondition, a well conducting electrolyte for the flux of ions must be available. Third, there must be a metallic connection between the anodic and cathodic reaction site for the flux of electrons. And finally, as the fourth requirement, a depassivation of the steel in the concrete must occur. The first three conditions are generally fulfilled for concrete bridges: First of all, oxygen is only exceptionally prevented from entering in the case of continuously submerged structures. In addition, water regularly has access to the reinforcement. And finally a metallic connection is usually provided by the reinforcement mesh. Consequently, the time of initiation ($T_i$) regularly depends on the time necessary for the depassivation of the steel in concrete. This depassivation either occurs because the carbonation depth ($x_c$) becomes equal to the concrete cover ($c$) or because a critical chloride level develops at the reinforcement ($C(c) = C_{Crit}$).

The main factors that influence the time to depassivation are the concrete cover ($c$), the exposition to chlorides and to carbon dioxide, the concrete material properties in terms of diffusion coefficients both for carbon dioxide ($D_{CO2}$) and chlorides ($D_{Cl^-}$), and finally both the critical chloride content that causes depassivation ($C_{Crit}$) and the amount of carbon dioxide necessary for causing carbonation of a unit volume of concrete.

The relevant literature provides many models for the time-dependent calculation of the carbonation depth and the chloride content (e.g. Böhni 2005: 22, 29; fib 2010b: 169–177; fib 2006). Each one of these models considers the above mentioned influencing aspects; however they considerably differ in terms of complexity.

In the scope of the present thesis, choice is made for two simple models, which are used by Pommerening et al. (2008: 17–28) for describing carbonation and chloride ingress. Their model for describing the chloride ingress uses the following simple equation 4.25 for the development of the front $x_{Cl,crit}$ where a critical chloride concentration of $C_{Crit}$ occurs.

$$x_{Cl,crit} = \sqrt{D_{eff} \times a_C \times t} \tag{4.25}$$

with

| | |
|---|---|
| $x_{Cl,crit}$ | depth [mm] at which $C = C_{crit}$ with $C_{crit} = 0.4\%$ for mild steel and $C_{crit} = 0.2\%$ for prestressing steel |
| $D_{eff}$ | effective chloride diffusion coefficient that accounts for the value of $C_{crit}$ as well as the exposition to chlorides and the local vulnerability to chloride diffusion in the form of the chloride diffusion coefficien $D_{Cl^-}$ |
| $a_C$ | coefficient that accounts for the average time of chloride exposition, e.g. $a_C$=0.25 (3 month/year) |

Values for the effective chloride diffusion coefficient $D_{eff}$ are provided by Pommerening at al. (2008: 19–23) for a variety of bridge specific exposition and vulnerability situations. In the scope of the present thesis, these values are condensed into three parameter value ranges, which describe high, medium and low (H/M/L) deterioration intensities. Each one of these three classes is defined by the minimum, the most likely, and the maximum values. The effective diffusion coefficient $D_{eff}$ differs between prestressing and mild steel because the critical chloride concentration $C_{Crit,}$ which is different for both steel types, is one of the determining factors of $D_{eff}$.

**Table 4-3:** Effective chloride diffusion coefficient for different deterioration intensity classes

| | L | | | M | | | H | | |
|---|---|---|---|---|---|---|---|---|---|
| | min | mod | max | min | mod | max | min | mod | max |
| Exposure and vulnerability | Sound concrete with low exposition to $Cl^-$ | | | (1) Sound concrete with moderately high exposition to $Cl^-$<br>(2) Cracked concrete with medium exposition to $Cl^-$, | | | Cracked concrete and/or honeycombing, both in combination with high exposition to $Cl^-$ | | |
| $a_c\, D_{eff}{}^a$ (tendons) | 1 | 30 | 50 | 50 | 250 | 450 | 450 | 1100 | 1750 |
| $a_c\, D_{eff}{}^a$ (mild steel) | 0.5 | 20 | 30 | 30 | 165 | 300 | 300 | 800 | 1300 |

[a] Estimate based on the data provided by Pommerening et al. 2008: 21–23 (with $a_c$=0.25)

The model that describes the carbonation process uses the following equation 4.26 for the development of the carbonation depth $x_c$:

$$x_c = \sqrt{Kt} \tag{4.26}$$

with

$K$ carbonation coefficient, which accounts for the local vulnerability (e.g. concrete material parameters and cracks)

Pommerening et al (2008) refer to the results of an investigation by Novak et al. (2003) which provides values for the carbonation coefficient $K$ for various bridge elements under consideration of local vulnerability due to damages. Based on these results, the following categorisation into three damage classes is conducted in the scope of this thesis:

**Table 4-4:** Carbonation coefficient for different for different deterioration intensity classes

| | L | | | M | | | H | | |
|---|---|---|---|---|---|---|---|---|---|
| | min | mod | max | min | mod | max | min | mod | max |
| Exposure and vulnerability | Sound concrete in wet surroundings | | | (1) Sound concrete in periodically wet surroundings<br>(2) Damaged concrete in wet surroundings | | | Damaged concrete in periodically wet surroundings | | |
| $K^a$ | 0.5 | 3 | 5.5 | 2 | 6 | 10 | 6 | 12 | 18 |

[a] Estimate based on the data provided by Pommerening et al. 2008: 24

In order to define the above described H/M/L-intensity distributions for the values of $T_i$ in the case of mild steel and prestressing steel, the initiation periods $T_i$ are calculated as the minimum for carbonation and chloride ingress by applying the equations 4.25 and 4.26. The following Table 4-5 provides data estimates for the initiation times, which are based on such calculations. It can be seen that carbonation only accounts for the low intensity deterioration intensity while the different exposure and vulnerability conditions with respect to chloride ingress occur at all three intensity levels.

**Table 4-5:** Initiation periods $T_i$ for prestressing and mild steel

| | **L** | | | **M** | | | **H** | | |
|---|---|---|---|---|---|---|---|---|---|
| | max | mod | min | max | mod | min | max | mod | min |
| Exposure and vulnerability | (1) Sound concrete with low exposition to $Cl^-$ (2) high carbonation intensity (mild steel) | | | (1) Sound concrete with moderately high exposition to $Cl^-$ (2) damaged concrete with medium $Cl^-$-exposition | | | Damaged concrete in combination with high exposition to $Cl^-$ | | |
| Mechanism | $Cl^-$ and $CO_2$ | | | $Cl^-$ | | | $Cl^-$ | | |
| $T_i$ (prestressing) | | | | | | | | | |
| $c = 80$ mm | >1000 | 213 | 128 | 128 | 26 | 14 | 14 | 6 | 4 |
| $c = 50$ mm | >1000 | 83 | 50 | 50 | 10 | 6 | 6 | 2 | 1 |
| $T_i$ (mild steel) | | | | | | | | | |
| $c = 40$ mm | >1000 | 80 | 53 | 53 | 10 | 5 | 5 | 2 | 1 |
| $c = 25$ mm | >1000 | 31 | 21 | 21 | 4 | 2 | 2 | 0.8 | 0.5 |

**Corrosion Rate ($\alpha_1$)**

In contrast to the elaborate differentiation with respect to the initiation time, Pommerening et al (2008) do not account for exposition and vulnerability dependent corrosion rates. Instead they consider a constant corrosion rate of 0.03 mm/a for chloride induced corrosion and of 0.003 mm/a in the case of carbonation induced corrosion.

However, investigations by other authors show that the corrosion rate also depends on the exposition and vulnerability situation. Relevant—and partly interdependent—parameters in this case are humidity, oxygen supply, concrete resistivity, and the temperature (TFB 2006: 7–11; Li et al. 2008: 1–2; Böhni 2005: 30–37; DuraCrete 1998: 23). A further decisive factor that controls the corrosion rate is found to be the depassivating mechanism—carbonation or chloride content (Tuuti 1982; DuraCrete 1998: 24–26). Also the degree of local concentration of the corrosive attack (*pitting*) has a major impact by practically multiplying the corrosion rate.

In the scope of this thesis, the corrosion rate is deduced from a time-dependent reduction $\Delta d$ of the reinforcement diameter. Pitting corrosion and effective time of wetness, during which corrosion takes place, is accounted for by the coefficients $\nu_{pitting}$ and $w_t$ as they are described by DuraCrete (1998: 29).

$$\Delta d\left(t\right)=\nu_{pitting}\,w_t k_s\,t \tag{4.27}$$

with

| | |
|---|---|
| $\Delta d$ | reduction of steel diameter |
| $\nu_{pitting}$ | coefficient for pitting. For homogenous corrosion $\nu_{pitting}=2;$ it can reach up to $\nu_{pitting}=4-10$ (DuraCrete 1998: 29; Strauss et al. 2009: 120) |
| $w_t$ | coefficient for time of wetness $(0\le w_t\le 1)$ |
| $k_s$ | rate of $\Delta d$ |

Example values of $k_s$, which account for exposition conditions and the depassivation mechanism, are provided by DuraCrete (1998):

**Table 4-6:** Corrosion rates $k_s$ and time of wetness $w_t$

| Exposure | $k_s$ μ | $k_s$ σ | $w_t$ |
|---|---|---|---|
| Carbonation induced | | | |
| Dry | 0 | – | 0.00 |
| Wet-rarely dry (unsheltered) | 0.004 | 0.003 | 1.00 |
| Moderate humidity (sheltered) | 0.002 | 0.001 | 0.50 |
| Cyclic wet-dry (unsheltered) | 0.005 | 0.003 | 0.75 |
| Chloride induced | | | |
| Wet-rarely dry (unsheltered) | 0.004 | 0.003 | 1.00 |
| Cyclic wet-dry (unsheltered) | 0.030 | 0.020 | 0.75 |
| Airborne sea water | 0.030 | 0.020 | 0.50 |
| Tidal zone | 0.070 | 0.040 | 1.00 |

*Source*: DuraCrete (1998: Table 2, p. 26) μ: sample mean value σ: standard deviation

The SAMS program accounts for corrosion by a reduction of the cross sectional area of the prestressing tendons ($A_p$) and of the shear reinforcement ($a_{sw}$). This is done by means of a reduction factor $\delta_{corr}$, which expresses the ratio of the remaining cross section to the initial cross section of the reinforcement—an approach that is also applied by Ahrens (2010: 38).

$$\delta_{corr}(t) = \frac{A_{corr}}{A_0} = \frac{d_{corr}^2}{d_0^2} = \frac{(d_0 - \Delta d)^2}{d_0^2} = \frac{d_0^2 - 2d_0\Delta d + \Delta d^2}{d_0^2} = 1 - \frac{2\Delta d}{d_0} + \frac{\Delta d^2}{d_0^2} \tag{4.28}$$

with

$A_0, A_{corr}$ initial and corrosion-reduced cross sectional area of steel

$d_0, d_{corr}$ initial and corrosion-reduced diameter of steel

By substituting equation 4.27 for $\Delta d$, the above equation 4.28 expands to:

$$\delta_{corr}(t) = 1 - \frac{2\nu_{pitting} w_t k_s t}{d_0} + \frac{\nu_{pitting}^2 w_t^2 k_s^2 t^2}{d_0^2} = 1 - \frac{\nu_{pitting} w_t k_s}{d_0}\left(2 - \frac{\nu_{pitting} w_t k_s t}{d_0}\right)t \tag{4.29}$$

The non-linear expression for $\delta_{corr}$ in equation 4.29 can be approximated by a linear term with the slope $\alpha$ in order to express the corrosion process in the form of equation 4.24. However, this linear approximation cannot be achieved by common linear regression because this may lead to starting values $\delta_0$ which are unequal to 1.0. Therefore a simplifying approximation is used by substituting the nonlinear curve (equation 4.29) by a simple secant that goes through the points that are defined by $\delta_0$=1.0 and $\delta_{sec}$=0.67. The latter value is justified based on the consideration that relevant $\delta$ values should be in the range between 1.0 and 0.67 because at higher deterioration levels structural safety is expected to be violated anyway if the original structure was economically designed. Based on these considerations, the deterioration rate $\alpha_1$ can easily be estimated by equation 4.30.

$$\alpha_1 = \frac{\nu_{pitting} w_t k_s}{d_0}\left(1 + \sqrt{\delta_{sec}}\right) \tag{4.30}$$

$$\delta_{p/asw}(t) = 1 - \alpha_1 t \tag{4.31}$$

with

$\delta_{sec}$ value of the secant endpoint for linear approximation ($\delta_{sec}$=0.67)

The following Table 4-7 contains an example evaluation of the linear approximation for the corrosion rate $\alpha$ which is given by equation 4.30. The three intensity classes (H, M, L) within the SAMS program are accounted for by specific value ranges for $k_s$ in analogy to Table 4-6. In this example, the high-intensity corrosion progress for prestressing tendons is accounted for by $k_s$-values in the range from 0.015 to 0.055, which represents the $\mu \pm \sigma$ value-range for cyclic wet-dry unsheltered conditions in Table 4-6 with an additional safety margin of 0.05. The high-intensity mild steel corrosion is assumed to be more severe because of higher exposure due to a lesser concrete cover so that a value range from 0.03 to 0.055 is assumed in this example. On the low-intensity side, the $k_s$-values are set according carbonation induced corrosion under moderate humidity (Table 4-6) in the range from 0.001 to 0.003 for the prestressing tendons, while the $k_s$-values for mild-steel corrosion are set to higher values from 0.002 to 0.01. Finally, $k_s$-values for the medium-range corrosion intensities are set in such a way that they fill the gap between the high and low value ranges. The coefficient $\nu_{pitting}$ is commonly set to a value of 6 for chloride induced corrosion in the case of high and medium corrosion intensities as well as in the case of the upper value for low-intensity corrosion. In the case of the lower bound and the mode value of low-intensity corrosion the coefficient $\nu_{pitting}$ is set to 2, which accounts for carbonation induced corrosion. The parameter $w_t$ is commonly set to 0.25, which means that wet conditions apply for a quarter of time. For mild steel reinforcement, the diameter $d_0$ is set to 14mm, which together with 12 mm and 16mm is a commonly used diameter for shear reinforcement; for prestressing strands a typical $d_0$-value of 15.7 mm is applied.

**Table 4-7:** Corrosion rates $\alpha_1$ for prestressing and mild steel

| | **L** | | | **M** | | | **H** | | |
|---|---|---|---|---|---|---|---|---|---|
| $\alpha$ in [‰] | min | mod | max | min | mod | max | min | mod | max |
| Exposure and vulnerability | (1) Sound concrete with low exposition to $Cl^-$, or (2) high carbonation intensity (mild steel) | | | (1) Sound concrete with moderately high exposition to $Cl^-$ or (2) damaged concrete with medium exposition to $Cl^-$, | | | Damaged concrete in combination with high exposition to $Cl^-$ | | |
| $k_s$ (prestressing) | .001 | .002 | .003 | .003 | .009 | .015 | .015 | .035 | .055 |
| $k_s$ (mild steel) | .002 | .006 | .010 | .010 | .020 | .030 | .030 | .0425 | .055 |
| $\alpha_1$ (prestressing) [a] | 0.06 | 0.1 | 0.5 | 0.5 | 1.6 | 2.6 | 2.6 | 6.1 | 9.6 |
| $\alpha_1$ (mild steel) [a] | 0.13 | 0.4 | 2.0 | 2.0 | 3.9 | 5.8 | 5.8 | 8.3 | 10.7 |
| Mechanism | $Cl^-$ and $CO_2$ | | | $Cl^-$ | | | $Cl^-$ | | |

[a] With $d_0$=14 mm for mild steel and $d_0$=15.7 mm for prestressing tendons, $w_t$ is uniformly set to 0.25 for all intensity classes. The coefficient $\nu_{pitting}$ is commonly set to a value of 6 for chloride induced corrosion in the case of *H* and *M* corrosion intensities as well as in the case of the upper value for *L* intensity. In the case of the lower bound and the mode value of *L* intensity $\nu_{pitting}$ is set to 2, which accounts for carbonation induced corrosion.

The linear approximation by equation 4.31 yields very accurate results compared to the formulation according to equations 4.28 und 4.29 as can be seen from following Fig. 4.12. Only for high deterioration levels with more than 50% loss, the approximation becomes too conservative. However, such deterioration levels should be beyond the practically relevant range.

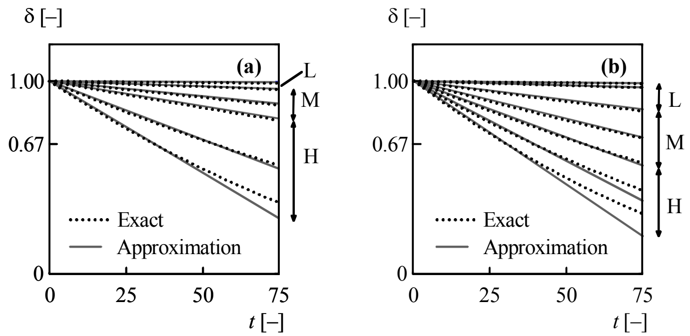



**Fig. 4.12:** Deterioration curves for (a) prestressing and (b) mild steel for the three intensity levels (H, M, L) calculated for the upper and lower bound as well as the mode of the deterioration rate $\alpha$ (Table 4-7). The dotted curves represent the exact equation 4.29 while the solid lines represent the linear approximation functions.

### 4.4.3 Deterioration of FRP Strengthening Systems

The deterioration of *fibre reinforced polymer* systems (FRP) is critically influenced by the decay of the bond to the concrete surface—but also the fibres and the polymer of the FRP strips and sheets are subject to deterioration (e.g. fib 2001: 109; Banthia et al. 2010: 47; Karbhari 2003: 244). However, there is only little quantitative information available on the long-term behaviour of FRP components for bridge strengthening. Data provided by Banthia et al. (2010) on an investigation of four Canadian bridges, which have been strengthened by FRP in the years from 1996 to 2001, show that the bond remained sound in most places. However, for each of the investigated bridges some locations with alarmingly reduced bond strength were found. In the case of one of the investigated bridges, it is assumed that the bond deterioration was caused by water, which leaked through the deck and eventually led to bond decay due to thawing and freezing action.

A simple model for estimating the long term degradation of FRP system properties—tensile strength of the FRP strips and the adhesive bond to the concrete—under consideration of environmental influences, has been developed by Karbhari et al. (Karbhari and Lee 2011, Abanilla et. al 2006). Their model is based on findings from accelerated testing and applies a transformation to a long term perspective; it is expressed by the below equation 4.32.

An experimental investigation of three different strengthening systems (Material ‘A’, ‘B’, and ‘C’) for specifying the parameters $F_{Env}$ and $A$ in the below equation 4.32, shows that the material-dependent coefficient $A$ considerably varies across the different systems (Wilcox 2008: 41): For the deterioration of the tensile strength of FRP strips, this coefficient ranges from 0.63 to 3.66 while for the tensile strength deterioration of the adhesive it ranges from 0.81 to 5.89. For the tensile strength of FRP strips, the influence of saline and / or alkali environmental conditions, represented by the factor $F_{Env}$, is found to be in the range of 0.67 (Material ‘C’) to 0.99 (Material ‘A’) for saline environments, while it ranges from 0.84 (Material ‘C’) to 0.93 (Material ‘A’) for alkaline environments.

$$P(t) = \frac{F_{ENV} P_0}{100}\left[100 - A\ln(t)\right] \tag{4.32}$$

with

| | |
|---|---|
| $P(t)$ | time dependent performance, e.g. tensile strength or modulus |
| $F_{ENV}$ | modification factor in order to account for environmental influences (salt water and alkali supply) |
| $P_0$ | initial performance value |
| $A$ | material-dependent coefficient |

The below Fig. 4.13 illustrates the deterioration curves for the tensile strength of three different strengthening materials (A, B, C) with variations of the environmental influences on degradation ($F_{Env}$).

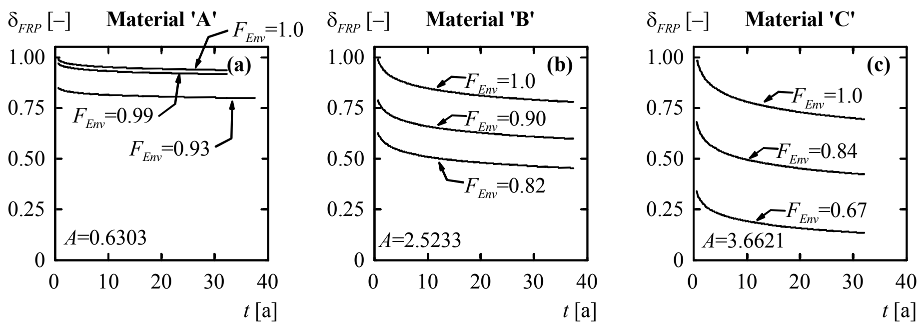



**Fig. 4.13:** Deterioration of tensile strength (FRP-strips) for three different strengthening systems according to Wilcox (2008: 41–45): (a) Degradation of material system ‘A’ ($A$=0.6303); (b) Material system ‘B’ ($A$=2.5233); (c) Material system ‘C’ ($A$=3.6621). The environmental influences $F_{Env}$, as they are found by Wilcox (2008: 45), are specified in terms of three exposure types: no influence ($F_{ENV}$=1.), alkaline environment (lowest value for $F_{ENV}$), and saline environment.

### 4.4.4 Development of Concrete Strength (Deterioration and Increase)

**Introduction**

There are two opposite developments of the concrete strength, the first is a decrease because of deterioration mechanisms and the second is an increase due to continuing hydration. Both effects are accounted for in the SAMS program.

**Deterioration of Concrete Strength**

Deterioration of concrete, which can result in loss of compressive strength, is typically caused by freeze thaw, external chemical attack (e.g. sulphates), and internal chemical reactions such as alkali-aggregate reaction (AAR) (fib 2008a: 42). Of these mechanisms, only the latter (AAR) is of greater relevance for bridges, while freeze-thaw is mostly relevant for dams or car parks and chemical attack naturally occurs for industrial floors, sewage plants, chimneys, as well as tanks and pipes (fib 2008a: 43).

The documentation '*Alkali-Aggregat-Reaktion (AAR)'* of the Swiss Federal Roads Office names three main categories of factors that influence the alkali-aggregate reaction (ASTRA 2007: 20–34):

- *concrete properties*, e.g. reactivity of aggregates, cement type, and pore water alkalinity
- *environment*, e.g. water and alkali supply (e.g. chlorides), wet/dry cycles, and temperature
- *structure*: e.g. dimensions of members, prestressing force, position and orientation

The first influence category describes the general susceptibility of the concrete to AAR; this category relates to the extent to which AAR actually affects the concrete strength and with which order of magnitude the AAR proceeds if beneficial conditions (e.g. water and alkali supply) occur. The remaining two influence categories both govern the speed of the reaction within the range that is defined by the concrete susceptibility.

However, in contrast to steel corrosion, the relevant literature does not provide commonly accepted models for the description of alkali-aggregate-reactions in terms of speed and effects on material properties, especially regarding concrete strength. Instead, the literature mainly provides results of isolated investigations with often significantly differing results regarding the effects on concrete strength.

As a solution to this problem, a simple approximate model is developed in the scope of this thesis for describing both the speed and the effect of AAR. Yet, it must be kept in mind that this model is not validated and its main purpose is to deliver plausible estimates for describing the alkali-aggregate reaction in quantitative terms. For more accurate bridge specific modelling of the AAR influences, this simple linear model can be adjusted to inspection results (see section 4.6).

The simple approximate model is developed on the basis of two data sources: One is the investigation of McColm and Huynh (ASTRA 2007: 19), which reports on the long-term AAR-induced strain development in concrete specimens. The other source is the established relationship between AAR-induced strain and the residual mechanical concrete properties, such as compressive strength, as it is reported by the ISE in 1992 (ISE 1992: 14; ASTRA 2007: 34).

First, the time dependent development of the AAR induced strain ($\varepsilon$) is described by a trilinear relationship as depicted by the approximation curves in Fig. 4.14 (a). The curves (1) and (4) in this figure are tri-linear approximations of the data reported by McColm and Huynh (ASTRA 2007: 19) for different concrete mixtures, where the mixture (1) is certainly much more susceptible to AAR than mixture (4). The two other curves (2) and (3) are weighted averages of the curves (1) and (4). These approximation curves are transformed with the help of a linear δ-ε relationship that expresses the ratio δ of the uniaxial compressive strength to the 28-day strength as a function of AAR induced strain (ISE 1992: 14) as it is depicted in Fig. 4.14 (b). This transformation yields the tri-linear δ-*t* relationships that are depicted in Fig. 4.14 (c).

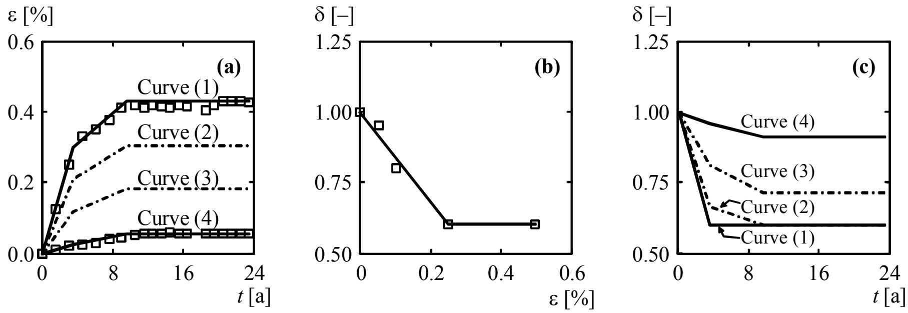



**Fig. 4.14:** Development of an approximate model for describing time-dependent AAR effects on concrete compressive strength: (a) Strain-time relationship in terms of a linear approximation (curves (1) and (4)) of data reported by McColm and Huynh (ASTRA 2007: 19) for different concrete mixtures and hypothetical intermediate linear courses displayed by curves (2) and (3); (b) Ratio of uniaxial compressive strength to the 28-day strength as a function of AAR induced strain (ISE 1992: 14); (c) Approximate δ-*t* relationship for the curves (1)–(4).

For actual SAMS model-definition, the following stepwise procedure should be applied:

- As a first step, the specific mean (or modal) values for the parameters $\delta_{k1}$, $\delta_{k2}$, and $T_i$ as well as $\alpha_1$ and $\alpha_2$ are defined in order to account for the influences of the first category (*concrete properties*). This may for example be achieved by choosing curve (1) of the above Fig. 4.14 (c) with a highly AAR-susceptible concrete mix.
- As a second step, the value ranges around the mean/modal values are defined in order to account for the influences of the second and third category (*environment* and *structure*), which both refer to the intensity spectrum of AAR due the external conditions.

**Increase in Concrete Strength**

In absence of deterioration processes that weaken the compressive strength, continuing hydration leads to a considerable increase of concrete strength (e.g. Wesche 1993: 65–70; 218–221). In the scope of the present thesis, this is accounted for by applying the approach that is provided by the Draft Model Code 2010 (fib 2010a: 128). This approach uses the equations

4.33 and 4.34 for estimating the time-dependent development of the mean compressive strength $f_{cm}(t)$ with the help of.

$$f_{cm}(t) = \beta_{cc}(t) f_{cm} \tag{4.33}$$

$$\beta_{cc}(t) = e^{s\left(1-\sqrt{\frac{28}{t}}\right)} \tag{4.34}$$

with

$f_{cm}$ mean compressive strength at 28 days $\left(f_{cm} = f_{ck} + 8MPa\right)$

$t$ concrete age (with respect to the temperature during curing) in days

$s$ coefficient that accounts for the cement characteristics

= 0.38 for cement type 32.5 N

= 0.25 for cement type 32.5 R and 42.5 N

= 0.20 for cement type 42.5 R, 52.5 N, 52.5 R

The following Fig. 4.15 shows that the compressive strength $f_{cm}$ considerably increases compared to the strength at 28 days, for example by 22% (cement type 42.5 R) or even by 46% (cement type 32.5 N). The figure also shows that this increase mainly occurs in the first year after the pouring of the concrete. However, this increase in concrete strength must not be superimposed with a decrease due to AAR because the values that are reported by the ISE in 1992 (ISE 1992: 14; ASTRA 2007: 34) refer to the 28-day compressive strength (see Fig. 4.14).

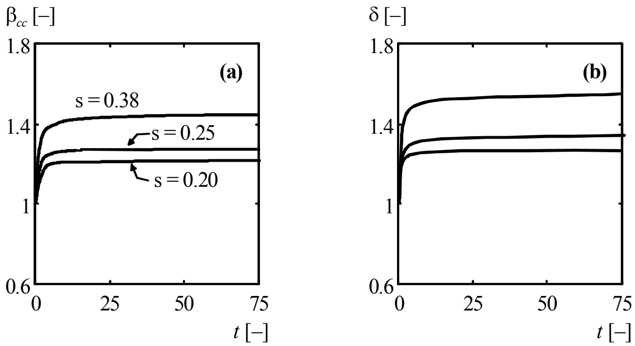



**Fig. 4.15:** Increase in compressive strength of concrete due to continuing hydration for different cement characteristics (s=0.38, 0.25, 0.20): (a) increase of $f_{cm}$ expressed by the coefficient $\beta_{cc}$ and (b) increase of $f_{ck}$=$f_{cm}$ − 8MPa expressed by the coefficient $\delta$=$f_{ck}(t)/f_{ck}(28\text{d})$ in the case of a concrete C40/50 with $f_{cm}$=48 MPa.

### 4.4.5 Deterioration of Concrete Surfaces

#### Overview

Deterioration mechanisms that affect the physical appearance of the concrete surface occur in many different forms. With particular regard to civil engineering structures and bridges, the German RI-EBW-PRÜF (2007, 2013) and Raina (1996: 10–34; 44–46) list several damage types, which can result from deterioration of the concrete. In the scope of the present thesis four characteristic groups of damage types are identified: *cracking*, *breaking*, *leakage related deficiencies*, and *dirt accumulation*.

- *Cracking* of the concrete occurs in many forms and due to many different causes. According to Raina (1996: 10–34) cracking can occur already during the construction phase because of early thermal or drying shrinkage, settlement of the hardening concrete, settlement of supports, and thermal movement. Later during the service life, further cracking can result from load effects such as overloading or from poor design, and also from deterioration mechanisms such as reinforcement corrosion, alkali-aggregate reactions, and freeze thaw.
- *Breaking* of the concrete matrix also occurs in several forms, as it is described by Raina (1996: 44–46): The most critical damage is spalling, which is defined as the breaking or chipping away of concrete fragments—possibly in connection with the exposition of reinforcing steel. This damage type is mostly caused by corrosion but it can also follow from structural overloading. A prestage to spalling is delamination in the form of horizontal fractures within the concrete (but without breaking off), which is usually caused by poor construction practice and corrosion. Less severe damages are popouts, which are conical cavities in the concrete surface due to breaking out of fragments and are usually caused by AAR or freeze-thaw. Further minor damages are near-surface defects, such as loss of surface mortar in the form of spread scaling or localised pitting, which are caused by freeze-thaw or poor construction practice, respectively.
- *Leakage related deficiencies* are found in the form of moist surfaces, rust stains and deposits on the concrete surface in the form of efflorescence, encrustations or stalactites, which result from dissolved minerals that are brought to the surface by leaching water (Raina 1996: 44–46).
- *Dirt accumulation* results, for example, from bird droppings, weathering, graffiti, and soot deposits at the bridge soffit.

#### Modelling of Concrete Surface Deterioration

The temporal evolution of concrete surface deterioration can be estimated by using mild steel corrosion as the reference deterioration mechanism. This is a valid assumption for three reasons. The first is that corrosion usually accounts for the most severe damages to the concrete surface—cracking, delamination and spalling—and therefore can reasonably be viewed as the leading deterioration mechanism. The second reason is that, except for dirt accumulation, all mechanisms which affect the concrete surface are controlled by the ingress of water. Thus, deterioration mechanisms other than reinforcement corrosion can be expected to proceed roughly parallel to corrosion. The third reason is that corrosion, of all deterioration mechanisms, is the most widely investigated mechanism and therefore numerical data are provided by the relevant literature for estimating the development over time. Notwithstanding this central role of reinforcement corrosion, the influences due to other effects—alkali-aggregate reaction, leakage related deficiencies, and dirt accumulation—can be accounted for by adapting the linear deterioration model to inspection results.

Rebar corrosion, according to the relevant literature (e.g. fib 2008a: 40–41; Li et al. 2008: 3–4; Böhni 2005: 72; Cairns et al. 2005), causes a characteristic deterioration process of the concrete surface (Fig. 4.16 (a)): No visible damage occurs during the initiation phase of this deterioration process, when depassivation due to chloride ingress and carbonation and initial corrosive attack of the reinforcement takes place. The second phase starts with the first occurrence of cracks at the time $t_{cr}$ (Fig. 4.16 (a)). This cracking results from the built-up of corrosion products at the interface between reinforcement and surrounding concrete, which causes tensile stresses in the concrete. The further course of this second phase is characterised by crack widening and by the occurrence of further cracks. Eventually, at the beginning of the third phase ($t_{sp}$, Fig. 4.16 (a)) the increasing built-up of corrosive products leads to delamination and spalling of the cover concrete. During the further course of this phase the condition worsens due to further cracking and spalling.

This deterioration progress can be approximated by a piecewise linear function as described in general form by equation 4.24. Value estimates for the parameters of this linear approximation ($T_i$, $\alpha_1$, $\alpha_2$, and $\alpha_3$ as well as $\delta_{k1}$ and $\delta_{k2}$) can be derived from the findings of an investigation, which has been conducted by the Swiss Federal Roads Office (TFB 2006: 45–47). This investigation of 27 inspection reports analysed data regarding the defect type (cracking or spalling), concrete cover $c$, reinforcement diameter $d_0$, and reduction of reinforcement diameter due to corrosion. Based on this analysis, two linear approximation functions could be established that describe the relationship between the ratio $c/d_0$ and the radial loss of reinforcement $\Delta r$ that is necessary to cause either cracking or spalling (Table 5.3 in TFB 2006: 47):

$$\Delta r_{cr} = 0.0004 + 0.0108 \frac{c}{d_0} \quad \text{for} \quad 0.5 \leq c/d_0 \leq 3 \tag{4.35}$$

$$\Delta r_{sp} = -0.001 + 0.168 \frac{c}{d_0} \quad \text{for} \quad 0.5 \leq c/d_0 \leq 3 \tag{4.36}$$

with

$\Delta r_{cr}, \Delta r_{sp}$ radial reinforcement loss necessary to cause cracking ($cr$) and spalling ($sp$)

In the scope of the present thesis, the times of first crack occurrence ($t_{cr}$) and spalling ($t_{sp}$) are estimated by applying the above equation 4.27, which describes the corrosive reduction of the reinforcement diameter, and considering the values for the initiation periods $T_{i,corr}$ for mild steel corrosion, as they are listed in Table 4-5. Thus, the times $t_{cr}$ and $t_{sp}$ can be calculated with the help of equations 4.37 and 4.38, which take into account that the crack and spalling inducing corrosion process initiates after the period $T_{i,corr}$.

$$t_{cr} = \frac{2}{\nu_{pitting}\, w_t k_s} \left( 0.0004 + 0.0108 \frac{c}{d_0} \right) + T_{i,corr} \tag{4.37}$$

$$t_{sp} = \frac{2}{\nu_{pitting}\, w_t k_s} \left( -0.001 + 0.168 \frac{c}{d_0} \right) + T_{i,corr} \tag{4.38}$$

with

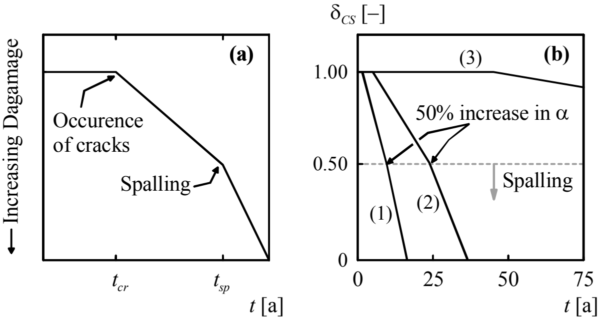
| | |
|---|---|
| $\nu_{pitting}, w_t, k_s$ | parameters describing exposure and vulnerability for corrosion |
| $T_{i,corr}$ | initiation period for mild steel corrosion |
| $c$ | concrete cover |
| $d_0$ | initial rebar diameter |

Accordingly, the parameters $T_i$ and $\alpha_1$ for the piecewise linear approximation of the concrete surface deterioration process are expressed by the following equations:

$$T_i = t_{cr} \tag{4.39}$$

$$\alpha_1 = \frac{\delta_{cracking} - \delta_{spalling}}{t_{sp} - t_{cr}} \tag{4.40}$$

As described above in sub-section 4.4.2, the values of the corrosion parameters $T_{i,corr}$, $\nu_{pitting}$, $w_t$, and $k_s$ depend on the exposure and vulnerability conditions and therefore can show different intensities. Consequently, also the initiation time for the concrete surface deterioration as well as the deterioration rates $\alpha$ can show different intensities. The following Fig. 4.16 (b) contains three example concrete surface deterioration curves that occur for three different corrosion intensities. In this example, the values $\delta_{cracking}$ and $\delta_{spalling}$ are defined as 1.0 and 0.5, respectively. An accelerated deterioration progress after the onset of spalling is estimated by a 50% increase in the deterioration rate.




| Curve | (1) | (2) | (3) |
|---|---|---|---|
| $T_{i,corr}$ [a] | 0.8 | 4 | 31 |
| $k_s$ [%] | 4.25 | 2 | 0.6 |
| $w_t$ [–] | 0.25 | 0.25 | 0.25 |
| $v_{pitting}$ [–] | 6 | 6 | 2 |

All curves: $d_0$=14mm
$c$=25mm

**Fig. 4.16:** Deterioration of the concrete surface: (a) General course of concrete surface deterioration as a result of rebar corrosion according to the relevant literature (e.g. fib 2008a: 40–41; Li et al. 2008: 3–4; Böhni 2005: 72; Cairns et al. 2005); (b) Three example courses (1) to (3) for different corrosion intensities.

### 4.4.6 Deterioration of Bridge Equipment

#### General Deterioration Model

The reviewed literature provides simplifying theoretical deterioration models for the bridge equipment on the basis of empirically collected data regarding the service life of typical bridge equipment elements (e.g. Haardt 2002: 32–35, Pommerening et al. 2008: 26–28). These models take the form of linear deterioration curves or, alternatively, the form of exponential curves. The linear form can be expressed by equation 4.41 and the alternative exponential form can be expressed, in analogy to Pommerening et al. (2008: 26), in terms of equation 4.42.

$$\delta(t) = 1 - \frac{t}{T_{SL}} \qquad \text{for} \quad 0 \le t \le T_{SL} \tag{4.41}$$

$$\delta(t) = \frac{1}{3}\left(4 - e^{(t/k)^p}\right) \qquad \text{for} \quad 0 \le t \le T_{SL} \tag{4.42}$$

with

$T_{SL}$ service life of equipment type (from empiric investigations)

$p$ coefficient, $p = 4$ according to Pommerening et. al (2008: 26)

$k$ coefficient, with $k = 0.922\,T_{SL}$ following from equation (4.42) and $\delta(T_{SL}) = 0$

In principle, the disadvantage of the simple linear approach (equation 4.41) is that it does not account for an initiation period in the form of a period where the equipment element performs very well, comparable to a new item. Moreover, the often observed increase in the deterioration rate during the later periods of the service life (e.g. Holst 2005: 27–28) is not accounted for by the linear approach. In contrast to this, the exponential model of equation 4.42 principally accounts for these two aspects as can be seen in Fig. 4.17 (a).

However, simple bi- and tri-linear approximation function can also be used as sufficiently accurate representations of the exponential model, as it occurs in Fig. 4.17 (b). This especially holds true for short service-life equipment, where even the simple bi-linear approximation is very close to the original model.

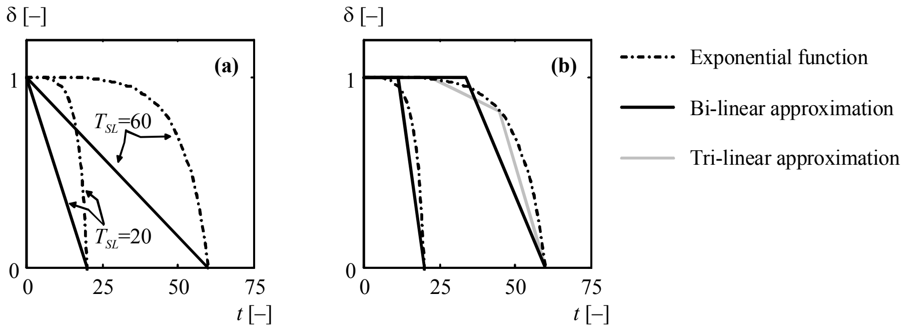



**Fig. 4.17:** Deterioration of bridge equipment: (a) Linear and exponential approaches for two example service life values; (b) bi- and tri-linear approximation functions.

An easy applicable linear approximation can be defined in such a way that specifying the value $T_{SL}$ suffices for calculating the time-dependent equipment deterioration. This is achieved by using a bilinear function that holds on to a value δ=1 until the exponential function, which it represents, reaches a value of 0.95 and then changes to a linear decrease until the end of the service life $T_{SL}$. Such a bi-linear approximation is illustrated in Fig. 4.17 (b).

**Asphalt Pavement**

The bridge pavement serves as a roadway for the traffic as well as a protection of the bridge structure against surface water and—by today standards—usually consists of three layers (Cziesielski 2006: 327–349; Mehlhorn 2007: 887–889). The first layer, which is applied directly to the concrete deck, is a waterproofing membrane, which is made up either of bitumen sheets or a resin sealing. The second layer is usually made of asphalt concrete or (stone) mastic asphalt and protects the waterproofing against loads from vehicles and temperature. Finally, the wearing course forms the top third layer and serves as the road surface; like the protective layer, it is made of asphalt concrete or (stone) mastic asphalt.

Deterioration of the asphalt pavement, which leads both to reduced serviceability and increased exposure of the structure to water and chlorides, occurs in the form of four different damaged types *surface defects*, *surface deformation*, *cracking*, and *potholes*. These damage types are described in detail by Walker (2002):

- *Surface defects* are erosive losses of pavement material. They occur in the form of *ravelling*, which is a progressive exposure of fine and coarse aggregate caused by ageing and poor construction practice. They also appear as *polishing* in the form of traffic induced abrasion of aggregates, which leaves a slippery road surface.

- *Surface deformations* occur in the form of rutting, which are channel-like depressions in the road surface along the wheelpaths caused by material displacement under traffic loads. Other forms surface distortions are (*frost-*) *heavings* or *shoving*, with the latter describing material displacements orthogonal to the traffic direction.
- *Cracking* of the asphalt occurs in many forms and due to many different causes. Cracks can occur in transversal or longitudinal direction as well as in the form of *map* (also: *alligator*) *cracking* with irregular and interconnected pattern. Cracks in the asphalt pavement are usually caused by traffic loads (fatigue), aging of the asphalt, and stresses from shrinkage or temperature.
- *Potholes* are damages to the pavement, which are left behind from breaking out of asphalt fragments. They are usually caused by traffic loading and fatigue and often develop from map cracking.

The $T_{SL}$ values for asphalt pavement on bridges, which are found in the relevant literature (Pommerening et al. 2008: 28; Haardt 2002: 32; Yanev and Testa 2000: C-5/7; König et al. 1986: 269) range from as short as 2–15 years to 20–35 years. The mean service life is found to be around 15 years.

**Bearings**

Bridge bearings transmit the loads from the superstructure to the substructure and must allow for the horizontal displacement and rotational movements according to the designed structural layout, which result for example from thermal expansion and contraction or load related deflections (e.g. Tonias and Zhao 2007: 427; Mehlhorn 2007: 894). The typical defects, as they are described by König et al. (1986: 206–211) and as they are also are listed as example defects by the German RI-EBW-PRÜF (2007, 2013), can be categorised into two groups: damages to the bearing integrity itself and restricted or excessive movements and rotations. These defects occur in special forms, according to the different types of bearings that are applied:

- *Integrity defects* typically occur in the form of steel corrosion, mechanical failure of bearing components, and ageing and embrittlement in the case of elastomeric bearings. Of these defects, steel corrosion is found to be the most prominent; here *roller bearings* are the most severely affected bearing type (König et al. 1986: 207), but also other types—*rocker bearings*, *sliding bearings*, *pot bearings*, and *spherical bearings*—suffer from corrosive attack. Typical mechanical failures can occur in the form of broken rollers and broken guidance components in the case of roller bearings and in the form of dislocated sliding sheets in the case of sliding plate bearings.

- *Restricted movement and rotation* can occur, for example, in the form of dirt and debris accumulation. It is also quite clear, that the above described integrity defects can cause restricted movement and rotation when a certain deterioration level is reached. This is especially the case for roller bearings, where cases of blocking due to corrosion have been reported (König et al. 1986: 207). Restricted rotation can also occur in the form of a reduced distance between the upper and lower bearing plate of pot bearings.

The expected bearing service life ($T_{SL}$) is reported to range from 15 to 25 years to as long as 40 to 80 years with mean values around 25 to 40 years. (Pommerening et al. 2008: 28; Haardt 2002: 32; Yanev and Testa 2000: C-5/7; König et al. 1986: 269).

**Expansion Joints**

The purpose of expansion joints is to bridge the gap that is located between the deck and the abutment or between adjoining deck segments for facilitating temperature and load induced expansions. In order to do so, the expansion joint hast to meet three general requirements (Braun and Bergmeister 2004: 279–280): First, it must fulfil its structural purpose by carrying the acting loads and allowing for the necessary movements with low resistance. Second it must be durable and watertight in order to prevent water and chlorides from entering the gap. And, as a third requirement, it must ensure sufficient ride quality for the traffic as well as limited noise emission. The typical defects, as they are described by König et al. (1986: 206–211) and as they are also listed as example defects by the German RI-EBW-PRÜF, can be categorised into three groups: damages to the expansion joint *integrity* including loss of watertightness, *restricted movements*, and finally *reduced ride comfort and increased noise emission* (RI-EBW-PRÜF 2007, 2013).

- *Integrity defects* occur in the form of mechanical failure and corrosive attack. The most prominent of these damages is found to be breaking or loosening of components or of the joint itself (König et al. 1986: 212)—for example wear of the sliding bearings or breaking of the lamella beams of modular expansion joints. It can also occur in the form of debonding and fatigue-induced plastic cracking of asphaltic expansion joints. Especially debonding and damages to the sealing can lead to a loss of watertightness and thus cause danger to the bridge superstructure in the form of water and chloride ingress. Corrosion, as it is reported by König et al. (1986: 212), mainly affects modular joints because in the past for these types no precautions against damage of the sealing were made and thus unprotected joist beams were vulnerable to corrosive attack by water and chlorides.
- *Restricted movement* is usually caused by accumulation of dirt and debris, which can even lead to bending and breaking of fingers in the case of finger joints. A failure of the expansion joint in the form of restricted movement may induce considerable stresses due to constrained thermal expansion into structure.

- *Reduced ride comfort and increased noise emission* can results, for example, from depression of the asphaltic plug joint, loosening of connecting bolts of sliding plate joints, and wear of the hinges and the link plates of roller shutter joints.

The expansion joint service life ($T_{SL}$), as it is reported in the relevant literature (Pommerening et al. 2008: 28; Haardt 2002: 32; Yanev and Testa 2000: C-5/7; König et al. 1986: 269) ranges from 5–20 to 30–40 years with mean values around 20 to 30 years. According to Haardt (2002: 32), the asphaltic joints show the shortest service life and modular or finger joints are found to be long lasting equipment, while the roller shutter joints show medium service life results.

**Drainage System**

The purpose of the drainage system is to collect the rainwater and convey it to appropriate discharge locations, which are usually the sewer system or a separate reservoir (Braun and Bergmeister 2004: 301). This is necessary in order to ensure trafficability, which can be impaired by water accumulating on the carriageway (Braun and Bergmeister 2004: 301, Raina 1996: 146). Also, all deterioration mechanism affecting the main structure—for example corrosion, alkali-aggregate reaction, cracking and spalling—are directly connected to the ingress of water and chlorides into the structure. Therefore, a functioning drainage system is of utmost importance for a durable and trafficable bridge structure.

The bridge drainage system consists of three major functional units: the inlets, the pipes, and the outlets (Braun and Bergmeister 2004: 302). Additional components contributing to the conveyance of water from the bridge are kerb channels, which transport the run-off to the drainage inlets, and drain holes in box girders for discharging interior water accumulations from condensation and leaking pipes. The typical defects, as they are also described by König et al. (1986: 206–211) and Raina (1996: 147) and the German RI-EBW-PRÜF (2007, 2013), can be categorised into two groups. The first group encompasses damages to *integrity* including leakages of the system. The second group encompasses *clogging* of drainage components.

- *Integrity defects* occur in the form of mechanical defects, typically in the form of loose or broken hangers as well as loose or broken inlet or manhole covers, or in the form of leaking inlets and pipe joints (König et al. 1986: 214; RI-EBW-PRÜF 2007, 2013). Furthermore, integrity defects can occur in the form of corrosive attack of both the pipes and other components. Substantial danger for the bridge structure can follow if these defects cause leakages so that water and chlorides can penetrate the main structure.
- *Clogging* typically affects drainage inlets and pipes but also the kerb channels. This can result from accumulated debris and rubbish but also from freezing water. Foremost, clogging reduces the efficacy of the drainage system—or even puts it out of service—and thus poses a threat to traffic safety. Secondly, complete clogging or reduced flow can accelerate

corrosion of the drainage components. Thirdly, entrapped water may freeze and lead to ruptured drain pipes, which can lead to considerable loading of structural members with water and chlorides—especially vulnerable in this context are the webs and the bottom slab of box girder bridges with internal drainage pipes.

The expected drainage system service life ($T_{SL}$) is reported to range from 10–15 years to 30 to 50 years with mean service life ranging from 22 to 35 years (Pommerening et al. 2008: 28; Haardt 2002: 32; König et al. 1986: 269).

## 4.5 Maintenance Activities

### 4.5.1 General Implementation of Maintenance Activities in SAMS

#### Overview

The present thesis takes a closer look at the following maintenance activities, which represent all four categories that are described in the above sub-section 2.6.1: *routine minor maintenance*, *special preventive activities*, *repair / replacement*, and *improvement*:

- *routine minor maintenance*
  - intensified routine maintenance
- *special preventive activities*
  - hydrophobic impregnation
  - coating of the concrete surface
  - local tendon and shear reinforcement conservation
- *repair / replacement*
  - concrete repair (girders, kerbs/sidewalks, substructure)
  - pavement resurfacing and replacement
  - bridge deck reconstruction
  - drainage system overhaul
  - expansion joint repair and replacement
  - bearing replacement repair and replacement
- *improvement*
  - bending and shear resistance improvement

The SAMS-program specifies maintenance activities in terms of ten maintenance parameters. These describe the intended impact in form of a temporal reduction of the deterioration progress ($v_1$, $v_2$) over a certain time span ($dt_1$, $dt_1$) and, possibly, a resulting element-condition improvement effect ($\Delta\delta$, $\delta_n$). Also side effects in terms of temporary traffic flow reduction ($\nu_{traf}$, $dt_{traf}$) and costs ($C_{app}$), and the maximum possible number of applications ($app_{max}$) are taken into account as maintenance parameters. These parameters, in analogy to the deterioration modelling, are also adapted to local peculiarities by means of intensity levels.

The maintenance parameters should be adjusted to bridge specific context, whenever possible. For this purpose, field data on maintenance effects and costs, for example from bridge inspections and from the commercial department, can be referred to. Otherwise, the values for the maintenance parameters must be estimated on the basis of the relevant literature or on the basis of educated guesses.

**Effects and Parameters of Individual Activities**

Each individual maintenance activity affects the bridge performance, especially in terms of condition and structural safety, in at least one of the following three ways:

- *delay* or even entire *prevention* of the onset of deterioration itself
- *reduction* of the deterioration rate, possibly even a temporary arrest to deterioration
- *improvement* of the actual condition and/or safety, which can occur in the form of incremental improvement, re-establishing the original state, or even upgrading beyond the initial level

The SAMS-program implements these maintenance effects by means of three *maintenance effect parameters* that modify the deterioration development. The first maintenance effect parameter is the reduction factor $\nu$, which accounts for a deterioration rate reduction ($0\leq \nu \leq 1$) and also for a delay of the deterioration onset or temporary arrest ($\nu$=1). In the scope of this thesis, a two-step reduction of the deterioration rate with two factors $\nu_1$ and $\nu_2$, which are effective for the successive periods $dt_1$ and $dt_1$, is taken into account. By doing so, a temporary arrest of deterioration ($\nu_1$=1.0) and a following reduced deterioration rate ($0<\nu_2<1$) can be modelled. The second maintenance effect, in the form of the increment $\Delta\delta$, describes the improvement of the condition level $\delta$ of the structural parameters. Finally, the third parameter $\delta_n$ describes an explicit value to which the $\delta$-level is set as a result of a maintenance activity; this can be, for example $\delta_n$=1 in the case of perfect replacement.

$$\alpha_{res} = \alpha(1-\nu) \tag{4.43}$$

$$\delta_{res} = \delta + \Delta\delta \tag{4.44}$$

$$\delta_{res} = \delta_n \tag{4.45}$$

with

| | |
|---|---|
| $\alpha_{res}, \delta_{res}$ | deterioration rate ($\alpha$) / level ($\delta$) that results from maintenance application |
| $\alpha, \delta$ | natural deterioration rate ($\alpha$) / level ($\delta$) (without maintenance effects) |
| $\nu$ | reduction factor (maintenance effect) |
| $\Delta\delta$ | increment (maintenance effect) |
| $\delta_n$ | reset value (maintenance effect) |

Within the SAMS algorithm, a maintenance activity is specified by six *maintenance effect parameters* $\Delta\delta$, $\delta_n$, $\nu_1$, $\nu_2$, $dt_1$, and $dt_1$. The time of application $T_{app}$, which must be known for calculating the explicit maintenance effect on the deterioration curve, is not a maintenance parameter in the closer sense, but rather a variable that results from the application of a certain maintenance strategy.

This method has a strong background in the relevant literature. For example, an analogous form is found in the works by Neves and Frangopol (2005) and Neves (2005), who use an additive reduction of the deterioration rate in the form $\alpha_{res}=\alpha-\nu$. Also Petcherdchoo et al. (2008) and Denton (2004) consider reduced deterioration rates during the effective period of the maintenance technique. A somewhat less direct way of dealing with reduced deterioration rates is found in Haardt 2002 (pp. 33–35), who considers a reduced deterioration rate in the form of an extended remaining component service life. In contrast to this, Pommerening et al. (2008: 38) do not account for a partial reduction of deterioration rates, but only consider a temporary stagnation of deterioration progress as a maintenance result.

Regarding improvement and repair activities, also the use of parameters comparable to an increment $\Delta\delta$ and a reset value $\delta_n$ is widely practiced. Neves and Frangopol (2005), Neves (2005), and Petcherdchoo et al. (2008) consider both an increment and a reset value for modelling the maintenance effects. In contrast, Pommerening et al. (2008), Denton (2004), and Haardt (2002) only consider the latter one.

A crucial aspect in the case of condition improvement activities is how these activities affect the post-intervention deterioration rate. In the scope of the present thesis it is assumed that the post-intervention deterioration rate $\alpha$ depends from the actual condition level $\delta$ after intervention. This approach is also applied by Beek et al. (2003: 257–259) for concrete structures. They label this method as *non-repeating* and oppose it to *repeating* deterioration curves, where the deterioration rate $\alpha$ after each maintenance activity '*is the same and equals the rate of deterioration* [...] *at time zero; in other words, after every lifetime extension the propagation curve repeats, as is the case for paint on steel*' (Beek et al. 2003: 257). Neves (2005: 77–79) follows a different approach as he applies a model where the deterioration rate increases with time and thus he chooses the post-intervention deterioration rate $\alpha$ as the rate at the time of maintenance application. The actual SAMS version implements the non-repeating approach because it corresponds with the assumed deterioration courses that is introduced in sub-section 4.4.1, which assume that the deterioration rate depends on the current deterioration level.

An example modelling of the maintenance effects is depicted in Fig. 4.18. The left hand diagram in Fig. 4.18 (a) shows the natural deterioration course with a change in the deterioration rate $\alpha$ at the level $\delta_{k1}$. The middle diagram in Fig. 4.18 (b) shows the effect a maintenance activity with an improvement by $\Delta\delta$ and two modifications of the deterioration rate and the resulting $\delta(t)$-curve: Directly after maintenance application the deterioration rate is reduced to zero ($\nu_1$=1); this is followed by a temporary reduction to 50% ($\nu_1$=0.5) which refers to the deterioration rate $\alpha_1$ while the deterioration level is above $\delta_{k1}$ and to the deterioration rate $\alpha_2$ while after the level exceeds $\delta_{k1}$. The right hand diagram Fig. 4.18 (c) depicts the deterioration rate $\alpha_{res}$, which results from superposition of the $\alpha$- and $\nu$-curves in the middle diagram Fig. 4.18 (b), together with the resulting $\delta(t)$-curve.

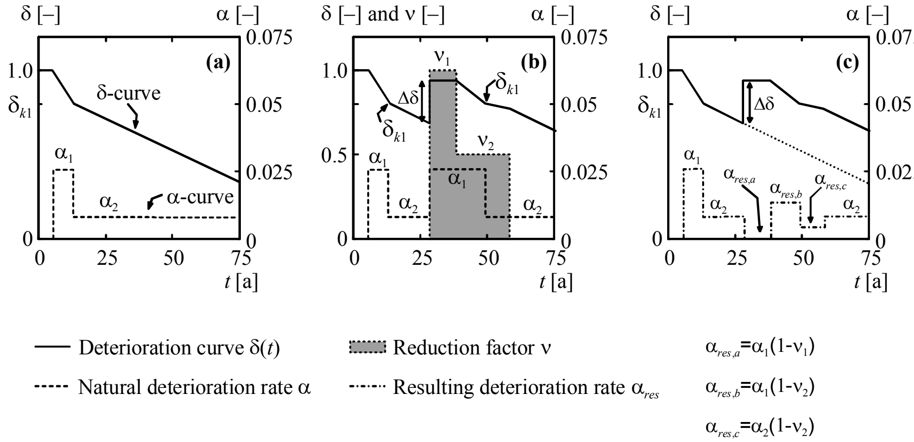



**Fig. 4.18:** Example modelling of maintenance effects: (a) Natural course of deterioration (solid line) and underlying deterioration rate $\alpha$ (dashed line); (b) maintenance effects $\nu$ and $\Delta\delta$ and basic deterioration rate $\alpha$ together with the modified $\delta(t)$ course; (c) resulting deterioration rate $\alpha_{res}$.

For the definition of the values for the maintenance effect parameters $\alpha$ and $\Delta\delta$ two important aspects must be taken into account. The first aspect is that a maintenance activity may affect several deterioration mechanisms simultaneously—both in different ways and in different intensities. This can be, for example, the case with the renewal of the asphalt pavement, which can be modelled in terms of a reset value $\delta_n$=1.0 for the pavement itself, while it also has additional beneficial effects in terms of reduced deterioration rates with respect the concrete strength and possibly also with respect to prestressing and mild steel corrosion in the deck slab. The second aspect is that these effects depend on the location within the bridge. For example, the repair or replacement of an expansion joint can have a significant effect on the corrosion of tendons and shear reinforcement close to the joint; however, this maintenance activity has no effect for the reinforcement in midspan. A further discussion on the background of these aspects can be found in Dette (2008). In order to account for such particularities, three intensity classes (H, M, L) for the value ranges of $\nu$, $\Delta\delta$ and $\delta_n$ are introduced in analogy to the formulation of the deterioration mechanisms.

In view of the various non-functional performance aspects that are considered for maintenance strategy assessment—economy, society, environment—further maintenance parameters must be defined used for describing the effects of the maintenance activities in terms of their costs, their societal aspects, such as disruptive effects on traffic flow, and further environmental effects, such as $CO_2$ emission and landfill production.

The present thesis considers the economic effects simply in terms of the cost per maintenance application ($C_{app}$). The traffic disruptions, which have societal as well as economic and environmental facets, are expressed in a different way by a coefficient $\nu_{traf}$ in analogy to the reduction of the deterioration rate. This reduction of the traffic flow is assumed to be constant and is described by a single value $\nu_{traf}$ that is effective over a period of $dt_{traf}$.

$$a_{res} = a\left(1-\nu_{traf}\right) \tag{4.46}$$

with

| | |
|---|---|
| $a_{res}$ | traffic availability, as it results from maintenance application |
| $a$ | availability (at the actual deterioration level) without maintenance effects |
| $\nu_{traf}$ | reduction factor (maintenance effect), effective for a period of $dt_{traf}$ |

In addition to this, a further technical parameter $app_{max}$ is introduced, which defines a maximum number of repeated applications of a maintenance activity. Environmental aspects, however, are not accounted for in the present version on SAMS for simplification reasons.

Altogether, each maintenance activity is described by a set of ten parameters, which consists out of six structural *maintenance effect parameters* ($\Delta\delta$, $\delta_n$, $\nu_1$, $\nu_2$, $dt_1$, and $dt_1$), two parameters describing the traffic-effect ($\nu_{traf}$, $dt_{traf}$), one economic parameter ($C_{app}$), and one technical parameter, which describes the maximum number of repeated application of the activity ($app_{max}$).

In analogy to the deterioration parameters, these maintenance effect parameters are also expressed in terms of distribution functions. In order to avoid odd parameter values, for example a worsened condition or an increased post-maintenance deterioration rate, the parameter distribution are specified in terms triangular or Beta-distributions. These distribution functions both are characterised by upper and lower bounds.

**Superposition of Maintenance Effects**

If the effects of different maintenance activities overlap, a superposition must be accounted for. In the relevant literature, two simple approaches are applied. The first is an *additive superposition* of the maintenance effects, where the reduction factors ν and the Δδ-values of the simultaneous maintenance effects are simply summed up. This approach is described by the following equation 4.47 (e.g. Neves and Frangopol 2005: 187; Neves 2005: 42–43; Kong and Frangopol 2003: 820). In contrast, the second *conservative superposition* approach (equation 4.48) only considers the maintenance effect with the highest impact on the deterioration rate and neglects all the other simultaneous maintenance effects (e.g. Neves 2005: 43–44).

$$\nu_{res} = \sum_j \nu_j \leq 1 \quad \text{and} \quad \Delta\delta_{res} = \sum_j \Delta\delta_j \qquad \text{(additive superposition)} \tag{4.47}$$

$$\nu_{res} = \max_j \left(\nu_j\right) \quad \text{and} \quad \Delta\delta_{res} = \max_j \left(\Delta\delta_j\right) \qquad \text{(conservative superposition)} \tag{4.48}$$

with

$\nu_{res}$ reduction factor resulting from superposition

$\Delta\delta_{res}$ incremental improvement resulting from superposition

$\nu_j, \Delta\delta_j$ maintenence effects of the $j^{th}$ maintenance activity

Also the SAMS program uses a simplifying superposition approach, which however allows for a more balanced superposition between these two extremes. This simplifying superposition method follows a *multiplicative approach* for the calculation of $\nu_{res}$ and as a consequence ensures a smoother superposition compared to the additive or the conservative approach. In contrast to this, the incremental improvements Δδ are superimposed by simple addition, assuming that each relevant activity influences the δ-level independently. Finally, the resulting reset value $\delta_{n,res}$ equals the maximum reset value that occurs for the simultaneous effective maintenance activities.

$$\nu_{res} = 1 - \prod_j \left(1 - \nu_j\right) \quad \text{and} \quad \Delta\delta_{res} = \sum_j \Delta\delta_j \quad \text{and} \quad \delta_{n,res} = \max_j \left(\delta_{n,j}\right) \tag{4.49}$$

As it can be seen in Fig. 4.19, the multiplicative approach can be interpreted as a compromise between the conservative and the additive approach. Here, the additive approach constitutes the upper bound for superposition, while the lower bound is expressed by the conservative approach that is applied by Neves (2005), where all but the most effective activities are neglected.

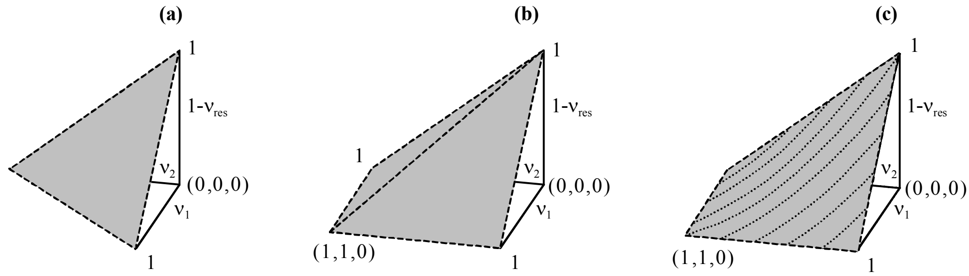



**Fig. 4.19:** Example superposition of two maintenance activities; the horizontal axes represent the reduction factors ($\nu_1$ and $\nu_2$) of the two activities and the vertical axes represent the resulting remaining fraction of the deterioration rate ($1-\nu_{res}$): (a) additive superposition: (b) conservative superposition; (c) multiplicative superposition (SAMS)

### 4.5.2 Intensified routine minor maintenance

This activity describes a set of regular routine minor maintenance measures, which are applied more intensively than usually in terms of shorter intervals and/or more care. The routine measures include sweeping of the pavement and cleaning of chloride affected elements, cleaning of drainage inlets and expansion joints, routine minor maintenance of moveable bridge equipment parts, crack seal and minor patching of the asphalt pavement (e.g. Vollrath and Tathoff 2002: 44).

Apart from the pavement, only those components and structural elements are considered to be affected whose deterioration is either caused or promoted by water and chloride ingress from leaking expansion joints, leaking asphalt sealing, and clogged drainage inlets. This is because routine minor maintenance works focus on the bridge carriageway. In detail, intensified routine minor maintenance is considered to reduce the rate of:

- pavement condition degradation
- prestressing tendon corrosion at the tendon high points
- shear reinforcement corrosion
- concrete strength degradation of the deck slab and the webs
- concrete surface condition degradation at the deck cantilevers, the kerbs, the webs, and the piers and abutments
- expansion joint degradation
- drainage system degradation
- bearing degradation

However, no data have been found that describe the economic impact. A rough estimate may be based on the data provided by Vollrath and Tathoff (2002: 44), who report an empirical value for the routine maintenance to costs of 0.56 €/m² annually. However, this still leaves open the question of the surplus costs for *intensified* routine maintenance works.

Also, no data for describing the effects of intensified routine maintenance on the deterioration rate and on the traffic flow have been found by a literature survey in the scope of the present thesis.

### 4.5.3 Hydrophobic Impregnation of Concrete

*Hydrophobic impregnation* is the application of chemical compounds in order to produce a water-repellent concrete surface. These compounds, usually silanes or siloxanes, penetrate the concrete up to a depth of 5 mm from the surface and lead to an internal hydrophobic coating—as opposed to filling—of pores and capillaries. The resulting water-repellent surface leads to a considerable reduction of water and chloride ingress and therefore a delay in the deterioration onset as well as to a reduction of the deterioration rate of steel corrosion and concrete deterioration.

This surface protection method has the advantage of little or even no aesthetic effects because it does not produce a film on the surface in contrast to coating. It can also have beneficial effects on the aesthetic performance because impregnation can reduce dirt accumulation as well as staining by algae, moss and fungi. However, as a disadvantage, it may accelerate carbonation progress. Its application should be limited to sound structures with no or only first signs of corrosion and should not be applied after onset of considerable corrosion. Otherwise, a thorough concrete repair—including patch repair, crack filling and replacement of corroded reinforcement may become necessary beforehand. (Denton 2004: C.3; Mehlhorn 2007: 987–988; fib 2008a: 147; Tosun et al. 2008).

In the scope of the present thesis, hydrophobic impregnation is considered to reduce, or temporarily suspend, the rate of the following deterioration mechanisms:

- concrete surface condition degradation
- concrete strength degradation
- prestressing tendon corrosion
- shear reinforcement corrosion

Actual values for describing the effect of this maintenance activity, in terms of $\nu_{1/2}$ and $dt_{1/2}$ values are provided, for example, by Denton (2004: 47) and Pommerening et al. (2008: 36–38). However, their data estimates show differences. Denton, who investigates maintenance strategies at the example of concrete bridge crossheads, does not assume a temporary stop of the deterioration progress but uses ν-values around 0.81 for overall condition and around 0.50 for overall safety with an effective period *dt* in the range from 7.5 to 12.5 years. But it must be kept in mind that Denton's data refer to a special structural component, namely concrete bridge crossheads, and thus are rather object-specific and do not describe the effects on basic deterioration mechanisms such as general steel corrosion. In contrast to this, Pommerening et al.—whose data refer to processes of chloride ingress and the associated reinforcement corrosion—consider in their investigation a complete standstill to deterioration progress for a period of 6 years; however, they also report on observed effective periods as low as 3 years for hydrophobic impregnations.

In economic terms, the average costs for hydrophobic impregnation can be estimated to 9.50 € per m² of treated concrete surface according to Pommerening et al. (2008: 36). In contrast to this, the cost data considered by Denton (2004: 47, 49) cannot be used as a reference for two reasons. Above all, the cost values include indirect traffic related costs and, secondly, the maintenance costs are not specified in terms of €/m² but in terms of costs per crosshead.

The traffic flow restrictions during concrete treatment works, however, largely depend on the number of lanes that must be closed during maintenance works in relation to the total number of lanes as well as on the time needed for the works. Thus, the traffic effects must be estimated considering the actual bridge geometry.

### 4.5.4 Coating of the Concrete Surface

*Concrete coating* produces a continuous protective layer on the concrete surface. General descriptions of this maintenance activity can be found for example in the works of Mehlhorn (2007: 987–988), the fib (2008a: 147) and Tilly (2002: 307), which are referred to in the following. A coating system consists of a primer and the actual coating and is usually applied with a thickness of 0.1 mm to 0.5 mm. Typical coating materials are, for example, acrylate resins as well as weather resistant two-component resins and also polymer modified cement. Some of these materials even are able to bridge cracks in the concrete surface.

The main coating effect is to prevent the penetration of water, chlorides and other pollutants into the concrete. Thus, it delays initiation and slows down deterioration mechanisms such as corrosion and alkali-aggregate reaction—just like it is the case with hydrophobic impregnation. Furthermore, a coating can also be applied in order to increase the resistance to mechanical wear and to chemical attack and also to restore *'a uniform surface appearance when repairs or weathering has made the concrete unsightly'* (Tilly 2002: 307). However, this generally leads to a significant change in the appearance of the structure. In the scope of the present thesis, concrete coating is considered to result in a

- restoration of a sound concrete surface condition
- reduction of the concrete surface condition deterioration rate
- reduction of the concrete strength degradation rate

- reduction of the prestressing tendon corrosion rate
- reduction the shear reinforcement corrosion rate

The actual maintenance effect of concrete coating can, for example, be estimated in form of a standstill to chloride ingress and to the associated deterioration mechanisms, such as corrosion and strength reduction, for a period of 12 years (Pommerening et al. 2008: 36–38).

In economic terms, the average costs for coating can be estimated to 18 € per m² of the treated concrete surface according to Pommerening et al. (2008: 36). The economic data by Denton (2004) cannot be used as a reference, as it is explained above in the case of hydrophobic impregnation.

The traffic flow restrictions during concrete treatment works, in analogy to the hydrophobic impregnation works, must be estimated considering the actual bridge geometry.

### 4.5.5 Concrete Repair

The activity *concrete repair* encompasses all measures that are applied in order to restore a cover concrete, which is cracked, delaminated, spalled and possibly chloride-contaminated or otherwise deteriorated. Typical concrete repair measures are filling of cracks, removing of the deteriorated concrete cover (e.g. by high pressure water jetting), cleaning and preservation and possibly replacement of corroded reinforcement, and also the reapplication of concrete or repair mortars.

The main effect of this measure is the restoration of the cover concrete to its original physical condition. However, the newly applied repair mortars often provide a poor colour match with the original surface so that the aesthetic condition usually is not completely restored. In such a case the above described coating of the concrete surface may be applied in order to generate a uniform surface appearance. Also, for preventative reasons, a follow-up hydrophobic impregnation of the restored concrete surface may be applied instead of coating—however, without the beneficial aesthetic effects of the coating. Further effects that follow from concrete repair works are a temporary reduction in the rates of reinforcement corrosion and of the concrete strength deterioration because of reduced ingress of water and chlorides due to repaired cracks and other surface damage. In the scope of the present thesis, concrete repair works are considered to restore a sound concrete surface condition as well as reducing, or even temporarily suspending, the rate of

- concrete strength degradation
- prestressing tendon corrosion
- shear reinforcement corrosion

While the effect on the concrete surface itself, which appears in a restoration to a sound condition, is quite clear, the quantitative effects on safety-relevant strength reduction and corrosion may be estimated according to Denton (2004: 48)—in this case even irrespective of his specific reference to concrete bridge heads—who considers concrete repair to result in a temporarily halt to induced reduction of structural safety of around six years.

The economic impact of concrete repair works may be estimated according to König et al. (1986: 278) who report the costs to be in the range from 50 DM/m² to 500 DM/m² with a mean value of 220 DM/m² and the area m² referring to the of bridge deck. Pommerening et al. (2008) do not refer to this maintenance activity, and Denton (2004) can also not be used as a reference because of the reasons stated in the case of hydrophobic impregnation.

### 4.5.6 Kerb and Sidewalk Rehabilitation

The activity *kerb and sidewalk* rehabilitation describes in the scope of the present thesis the set of intensive maintenance measures that are necessary to set the condition of the kerbs and sidewalks back to as-new level ($\delta_n$=1). It may encompass even complete replacement.

Similarly to general concrete repair works, kerb and sidewalk rehabilitation can also reduce deterioration of the deck concrete and also steel corrosion in the deck slab because of reduced ingress of water and chlorides due to repaired cracks and other surface damage. The kerb and sidewalk repair may require sophisticated platforms or scaffolding.

The costs can, for example, be estimated to be around 285.75 €/m² based on the estimate by Pommerening et al. (2008: 36), which refers to the actually rehabilitated kerb and sidewalk area. Additionally, reference may be made to König et al. (1986: 278) who report the costs to be in the range from 80 DM/m² to 130 DM/m² with a mean value of 110 DM/m² , in relation to the of bridge deck area.

### 4.5.7 Pavement Resurfacing

*Pavement resurfacing* is defined as the application of a thin asphalt overlay on the existing wearing course with the aim of restoring the initial road condition. However, the applicability of this measure is restricted to the repair of localised damages to the wearing course and to the repair of small defects of the protective layer. The main effect of this measure is the restoration of a good trafficability. In addition, it reduces the exposition of the bridge deck to water and chlorides by forming a water resistant surface; however, due to wear this resistance to water ingress is only a temporary effect. (Mehlhorn 2007: 990)

In the scope of the present thesis, the application of pavement resurfacing is considered to restore the original condition of the asphalt surface. It is also considered to slow down prestressing tendon corrosion at the high points and concrete strength degradation in the deck slab because of reduced water and chloride ingress. While the first effect to the pavement itself is simply expressed in terms of a condition reset value of $\delta_n$=1, empirical values for describing the latter effects in terms of the deterioration reduction factors $\nu$ to corrosion and strength reduction and their duration for have not been found by the literature survey.

Naturally, resurfacing activities require temporary restrictions to traffic. The actual extent, however, depends on the carriageway dimensions, such as the number of lanes that need to be temporarily closed in relation to the total number of lanes. As economy is concerned, Pommerening et al. (2008: 36) provide cost estimate value of 11 €/m².

### 4.5.8 Pavement Replacement

The activity *pavement replacement* describes the total replacement of at least the top two pavement layers—wearing course and protective layer—and possibly also the replacement of the waterproofing membrane, which is the third layer of the pavement. First the wearing course and the protective layer are usually removed by road milling machines, then the waterproofing membrane is cleaned and checked for integrity. After possibly replacing the waterproofing membrane—including surface preparation—the new protection layer and wearing course can be installed.

The total pavement replacement leads to a restoration of the original condition of the asphalt surface ($\delta_n$=1) as well as to a reduction of prestressing tendon corrosion at the tendon high points and a reduction of concrete strength degradation in the deck slab. However, in contrast to resurfacing, the reduction of water and chloride ingress is longer lasting, since not only the top layer but also the waterproofing membrane itself is restored to proper condition. Nevertheless, as it is the case with the above described pavement resurfacing, the literature survey did not succeed in finding empirical values for the intensity and duration of this effect.

The economic impact can be estimated according to Pommerening et al. (2008: 36) who consider a value of 60 €/m² with reference to the pavement area and according to König et al. (1986: 250–282), who find a range from 140 DM/m² to 160 DM/m², referring in contrast to the bridge deck area, on a time basis of 1985.

Pavement replacement requires temporary restrictions to traffic, depending on the percentage of lanes that have to be closed for rehabilitation works.

### 4.5.9 Bridge Deck Reconstruction

The maintenance activity *bridge deck reconstruction* encompasses a thorough replacement of deck concrete that is either deteriorated, for example by alkali-aggregate reaction, or contaminated by chlorides. It also encompasses the conservation or replacement and of corroding reinforcement. This is achieved by the following working steps (e.g. Vockrodt et al. 2003: 52–53; Tilly 2002: 331–332)

- removal of the deteriorated concrete layer by high pressure water jetting
- treatment of the corroded reinforcement and preparation of the exposed concrete surface
- if necessary: application of shear connectors and mild steel reinforcement
- application of a new concrete layer, which also can be newly designed as an *overslab*

Obviously, such major bridge deck reconstruction works necessitate the replacement of the asphalt pavement, while the above described pavement replacement works do not necessarily need to be combined with major concrete replacement. If the deck condition is fair enough, the pavement rehabilitation works can also be conducted with only minor preparation of the concrete surface.

The replacement of the deteriorated deck concrete has two major effects. On the one hand it results in a reduction of further deterioration. On the other hand it leads to a (partial) restoration of the bending behaviour, or even an improvement beyond the initial condition. The reduction of the corrosion rate and of the concrete degradation progress is achieved by replacing the chloride-contaminated concrete and by the cleaning and conservation of the corroded reinforcement. The restoration of the bending behaviour, in terms of bending stiffness and load bearing capacity, results from re-applying sound concrete in the compression zone. In the case of an overslab, which means that the new deck has a greater depth than initially, this can even lead to an improvement beyond the initial condition because of the increased effective depth. However, as it is the case with FRP-strengthening (Fig. 4.20) it must be kept in mind that the newly applied concrete layer is usually only effective for the loads that are applied after rehabilitation works—if not special measures such as temporary lifting of the structure before deck replacement are applied.

Analogously to the above described maintenance activities, empirical data on reduced deterioration rates of concrete and reinforcement, which stem from reduced water and chloride ingress, have not been found by the literature survey.

It is quite clear that this activity leads to significant and lengthy traffic reductions, depending on the extent of lanes worked on at the same time. However, no empirical economic data could be extracted for this special activity.

### 4.5.10 Drainage System Overhaul

This activity describes a major repair of the drainage system, including the complete replacement of the drainage pipes, with the objective to restore the drainage system to as-new condition.

Consequently, this activity puts the condition of the drainage system back to deterioration level of $\delta$=1. In addition, this restoration reduces the deterioration rate at those locations where defects are caused by a deficient drainage system. This can be, for example, corrosion of the shear reinforcement and of the prestressing tendons as well as alkali-aggregate reaction where a leaking drainage pipe connection causes water and chloride exposition of the web concrete. Also a major cause for tendon corrosion at the low points in mid span of box girder bridges can be eliminated by repairing leaking pipes: At these locations often poor concrete compaction occurs around the tendons (Zilch and Weiher 2007; König et al. 1986: 200–201) and thus water from leaking pipes accumulating on the bottom slab at the intersection with the webs poses considerable danger. In contrast to this, the deterioration processes in the deck slab are only affected to a very low extent by the drainage system overhaul because possibly leaking inlets only result in local deterioration.

An overhaul of the drainage system is assumed to cause no significant traffic disruption. This is especially true for box girder bridges with internal pipes. The costs can be estimated according to Pommerening et al. (2008: 36) who consider a value of 29 € per m² of the bridge deck surface and according to König et al. (1986: 250–282), who report value ranging from 50 to 150 DM/m² on a time basis of 1985.

### 4.5.11 Expansion Joint Repair and Replacement

The replacement of an expansion joint results in a restoration of the expansion joint to as-new condition ($\delta_n$=1); analogously the repair effect can be expressed in terms of an improvement $\Delta\delta$. Moreover, the repair and replacement activities considerably reduce concrete surface and strength deterioration as well steel corrosion in the vicinity of leaking expansion joints.

The maintenance effects of this activity can be estimated under consideration of the data estimates of Denton (2004: 47), who considers a reduction in the deterioration rates by 50% both for condition and safety related deterioration processes for a period ranging from 5 and 10 up to 25 years, during which the expansion joints do not leak.

The replacement of the expansion joints causes considerable traffic disruptions which depend on the time needed for repair and replacement works. For expansion joint replacement, the duration can, for example, be estimated on the basis of an empirically based value of 22 h/m referring to the expansion joint length (Pommerening et al. 2008: 36).

The costs for joint replacement can be estimated according to Pommerening et al. (2008: 36), who consider 1860 €/m. Also König et al. (1986: 254) provide price-data for the *replacement* of expansion joints of four German major valley bridges, which range from to 4192 €/m to 8409 €/m and at the price level of the year 2008. However, empirical cost values for expansion joint *repair* works are not reported in the reviewed literature.

### 4.5.12 Bearing Repair and Replacement

*Bearing replacement* restores an as-new condition of the replaced bearings ($\delta_n$=1) ); analogously the *repair* effect can be expressed in terms of an improvement $\Delta\delta$. Additional effects on other bridge components are not considered in the scope of this thesis.

Bearing replacement and repair works can cause traffic disruptions because of temporary bridge closures during maintenance works whose duration may be estimated at 15 h/bearing according to Pommerening et al. (2008: 36). The costs for bearing replacement can be estimated according to Pommerening et al. (2008: 36), who consider 2250 €/bearing. Alternatively the data by König et al. (1986: 280) can be considered, who report on replacement costs for bearings around 70 to 90 DM/m² with reference to the bridge deck surface.

### 4.5.13 Local Tendon and Shear Reinforcement Conservation

The activities *tendon conservation* and *shear reinforcement conservation* describe the thorough treatment of locally corroded tendons (or shear reinforcement) in the form of uncovering, anti-corrosive treatment, and sealing with protective repair mortar.

This measure only has a local effect in terms of a temporary reduction of the corrosion rate and does not affect any other deterioration mechanism. If truck-mounted platforms or special scaffolding are used for conservation works at the outside of the bridge, this activity causes traffic disruptions which may be estimated according to the ratio of one closed lane to the total number of traffic lanes.

### 4.5.14 Bending Resistance Improvement

The following three methods can be applied for improving the bending resistance (e.g. Buschmeyer et al. 2011; fib 2008a: 148; Mehlhorn 2007: 993, 1001; Vockrodt et al. 2003: 52–58):

- *additional mild steel reinforcement*, which is installed either together with a concrete *overslab* on the existing bridge deck, or which is embedded in shotcrete at the bottom or the webs of the bridge
- *additional bonded strips or plates*, which are made of fibre reinforced polymer (*FRP*) or in the form of steel plates—these can be either externally bonded to the concrete surface or, in the case of FRP strips, these can be embedded into slits that have been cut into the concrete for this purpose
- *external longitudinal post-tensioning* by externally applied tendons, which can be applied straight and in deviated form

In the scope of the present thesis, longitudinal bending resistance improvement is considered exclusively with respect to the first two methods. The strengthening effect of such an application is already accounted for in the equations 4.5 and 4.7, which are used to express the bending resistance $M_{Rd}$ (sub-section 4.3.3). Here the strengthening effect depends on the two parameters $A_{st}$ and $\sigma_{st}$ which stand for the cross sectional area of the strengthening components (mild steel plates or FRP strips) and the stress in the strengthening component under ultimate loading (Fig. 4.20 (a) and (b)). Crucially, the stress $\sigma_{st}$ depends on the strain increase ($\Delta\varepsilon_{st}$) under ultimate loading (e.g. Vockrodt et al. 2003: 54; Tilly 2002: 327–328, Onken and vom Berg 2001; fib 2001: 34–38). This strain increase is defined as the difference between the strain during the installation of the strengthening and the strain at ultimate loading (Fig. 4.20 (c) and (d)).

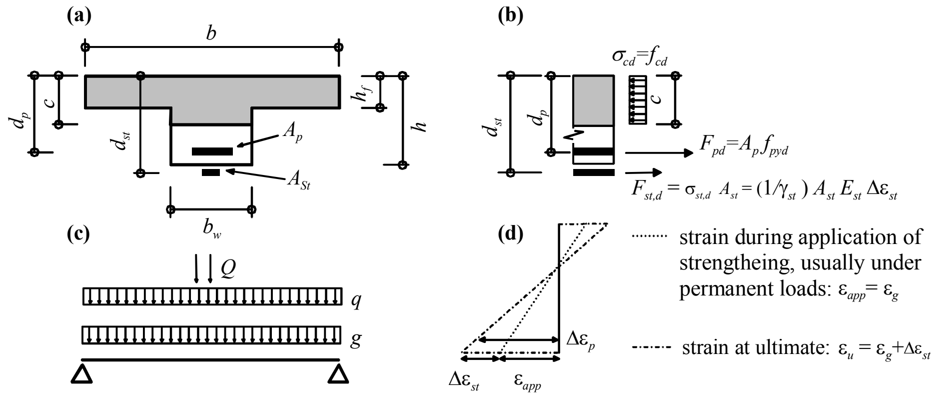



**Fig. 4.20:** Improvement of the bending resistance by application of additional reinforcement: (a) cross section with additional reinforcement $A_{St}$ applied to the bottom; (b) internal forces, including the contribution $F_{St,d}$; (c) example loading conditions with permanent loads *g*, which are acting during strengthening works, and life loads *q* and *Q*; (d) strain distribution.

In order to simplify the calculation, an approximation can be applied by assuming a defined stress $f_u$ (e.g. yielding or rupture) in the component. However, it must be kept in mind that this simplification is on the unsafe side in the case of high reinforcement ratios where failure is governed by crushing of the concrete and also in the case that steel yielding occurs before FRP fracture.

Nevertheless, in the scope of the present thesis, the approximation approach by assuming the stress $f_u$ at ultimate strain $\varepsilon_u$ is applied (e.g. Rostásy et al. 1996: 564–565). In the case of FRP strips, these values are calculated according to Mehlhorn (2007: 999) by taking into account the influence of the material properties, debonding (peeling-off), and the installation conditions (equation 4.50).

$$f_{ud} = \frac{1}{\gamma_{FRP}\gamma_1\gamma_a} E_{FRP}\varepsilon_u \tag{4.50}$$

with

| | |
|---|---|
| $\gamma_{FRP}$ | partial safety factor for FRP material properties: $\gamma_{FRP} = 1.2$ |
| $\gamma_1$ | partial safety factor for danger of premature debonding<br>$\gamma_1 = 1.0$ if premature debonding is not expected<br>$\gamma_1 = 1.4$ danger of premature debonding due to concrete strain |
| $\gamma_a$ | partial safety factor for application situation<br>(e.g. $\gamma_a = 1.2$ in the case of application by trained personel) |
| $E_{FRP}$ | Young's modulus, according to fib (2001: 9)<br>170 GPa for low modulus FRP and 300 GPa for high modulus FRP |
| $\varepsilon_u$ | ultimate strain, according to fib (2001: 9)<br>$\varepsilon_u = 1.6\%$ for low modulus FRP and $\varepsilon_u = 0.5\%$ for high modulus FRP |

Bending resistance improvement is assumed to cause some minor traffic disruptions from installation and removal of scaffolding and also from delivery of construction material. These disruptions may not occur in the case of box girder bridges, where the strengthening works can be conducted from within. Also some minor traffic disruptions are caused from weight restrictions during application in order to reduce the life load effects on the bridge. However, it is sufficient to pose those weight restrictions only to heavy trucks, so that the usual traffic can flow freely even during strengthening works (Rostásy et al. 1996: 548).

### 4.5.15 Shear Resistance Improvement

The relevant literature describes the following methods (e.g. Buschmeyer et al. 2011; fib 2008a: 148; Mehlhorn 2007: 993, 1001; Sigrist et al. 2006; Vockrodt et al. 2003: 59–67):

- *additional mild steel shear reinforcement*, which is installed in drilled holes or in slits that are prepared by high pressure water jetting—optionally, this can be combined with concrete replacement or with widening of the webs by shotcrete
- *FRP reinforcement* in various forms, such as sheets, strips or 'U-jackets' (or 'U-wraps'); the latter one describes the case that the FRP sheet (or fabric) runs down one outside of girder, continues around the bottom of the girder and runs up the opposite outside of the girder—the reinforcement elements can be either externally bonded to the concrete surface or, in the case of FRP strips, they can be embedded into slits
- *local vertical prestressing of the webs* by externally applied prestressing tendons
- *change of the structural system and/or change of the internal load distribution* by measures such as additional external longitudinal prestressing or change of the support conditions by additional hammerheads or even additional steel frameworks, which carry parts of the shear load to the supports

In the present thesis, the improvement of the shear behaviour by changing the structural system or the load distribution is not further discussed. Only the first three strengthening methods are considered in terms of a simplifying approach, which considers the application of additional shear reinforcement. The strengthening effect of this measure is already included in equation 4.16 in the form of the product $a_{st}\times\sigma_{st}$, which contributes to the shear resistance $V_{Rd,s}$ of the reinforcement. In analogy to the above described improvement of the bending resistance, the stress $\sigma_{st}$ in the strengthening components is estimated either as $f_{yd}$ in the case of additional mild steel reinforcements or as $f_{ud}$ in the case of shear strengthening by FRP.

Also, an optional web-widening by shotcrete may be accounted for by an increment $\Delta\delta_{fc}$. Then, this increment $\Delta\delta_{fc}$ stands for an equivalent increment with respect to the concrete strength $f_{c,web}$:

$$\Delta\delta_{fc} = \frac{f_{ce,add,d}\,\Delta b}{f_{ce,0,d}\,b_w} \tag{4.51}$$

with

| | |
|---|---|
| $f_{c,add,d}$ | design compressive strength of the added concrete layer |
| $f_{c,0,d}$ | initial design compressive strength of the existing web |
| $\Delta b$ | thickness of the added concrete layer |

In addition to the improvements $\Delta\delta_{ast}$ and $\Delta\delta_{fc}$ this activity can also lead to an effective reduction of the deterioration rate when the newly applied shotcrete shows a lower deterioration rate than the existing concrete or if this additional shotcrete layer slows down the deterioration rate of the existing shear reinforcement.

Similar to the improvement of the bending resistance, the shear strengthening activities cause some minor traffic disruptions.

## 4.6 SAMS-Model Adjustment to Bridge Inspection and Assessment Results

The theoretical model that describes the deteriorating bridge and the maintenance activities within the SAMS program should be adjusted to field data—as far as possible—in order to produce reliable results in the bridge specific context. Comprehensive field data for adjusting both the deterioration models and the maintenance parameters are usually available from the reports of both the general and the major bridge inspections. Also, more specific data may be provided in the form of findings from special inspections, for example measurements of concrete strength from core tests including the interpretation of these inspection results and—possibly—predictions of the future deterioration development. The deterioration model parameters $T_i$, $\alpha$, and $\delta_k$ are simply adjusted in such a way that the model curve approximates the actually observed development. The physical maintenance effects in the form of the deterioration reduction factors $\nu$ can be deducted from the deterioration progress before and after maintenance application, while the $\Delta\delta$ and $\delta_n$ values for repair and replacement activities are usually self explanatory.

As a first step, the field data of bridge inspections, such as lists of defects (Fig. 4.21 (a)), are allocated to deterioration mechanisms—for example web concrete surface deterioration, as in Fig. 4.21 (b)—and then are transformed to $\delta$-values, which in the present thesis range from 1 (initial condition) to 0 (worst). In the case of quantitatively measurable parameters, such as the concrete strength, the $\delta$-index describes the numerical ratio of the actual parameter value to its initial value and thus usually ranges from 1 (initial condition) to 0 (complete loss). Thus, the basic transformation from field data to the $\delta$-index is simply achieved by dividing the actual value by the initial value. When improvement or strengthening activities are applied, even values >1.0 can occur. However, in the case of qualitative parameters, such as the condition of the concrete surface, the transformation methodology of the field data is not that direct. In the present thesis, the δ-index refers to a qualitative rating scale in the range from 1 (*excellent*) to 0 (*very poor*). For transformation, the scale is subdivided into five deterioration levels (*excellent* [1.0], *good* [0.75], *fair* [0.50], *poor* [0.25], *very poor* [0.0]), which are described in terms of characteristic defect types (Fig. 4.21 (b)). In the scope of the present thesis, these characteristic defect types are defined in analogy to the example defects listed in the defect catalogue (*Schadenskatalog*) of the German RI-EBW PRÜF (2007, 2013). This catalogue provides a comprehensive set of example defects of civil engineering structures and associated defect ratings with respect to structural safety, traffic safety, and durability. Like the δ-scale, the RI-EBW-PRÜF also uses a five level defect rating scale. The calculation of the resulting δ-value, which summarises several defect types (cracking, breaking, leakages and dirt accumulation in the case of the concrete surface condition), is achieved by a worst defect classification system (equation 4.52). Detailed guidance for interpreting inspection results and transformation of the findings to the 1–0 δ-scale is provided in Appendix II.

$$\delta_{\{Name\}} = \min_i \left( \delta_{defect,i} \right) \tag{4.52}$$

with

| | |
|---|---|
| $\{Name\}$ | name that indicates the structural parameter<br>(e.g. $cs, web$ for the web concrete surface) |
| $i$ | index that indicates the defect types<br>(e.g. $i = 1$ : cracking, $i = 2$ : breaking, $i = 3$ : leakage) |
| $\delta_{defect,i}$ | rating of the defect $i$ |

As a second step, an *as-observed* δ-curve is constructed from the δ-values of successive inspection periods (Fig. 4.21 (c)). From this δ-curve the deterioration parameters, for example $T_i$ and α, can be deducted. Also the maintenance effect parameters, such as ν and *dT*, can be estimated from these curves by comparing the slopes of the δ-curve before and after maintenance application.

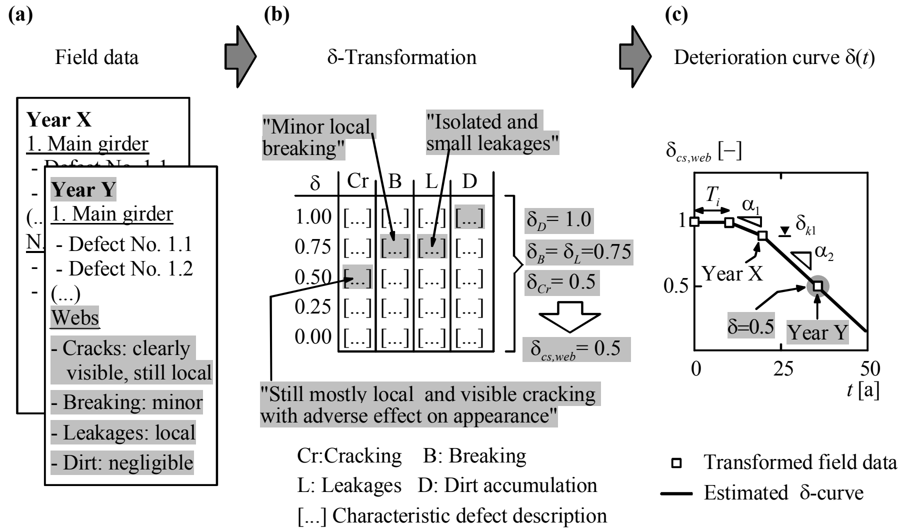



**Fig. 4.21:** Adjustment of deterioration parameters to field data, here illustrated with the example of web concrete surface deterioration: (a) field data in the form of reports of successive inspections (year X, year Y, ...); (b) transformation of field data to δ-values; (c) construction of the δ-curve and calculation of the deterioration parameters, such as $T_i$ and α.

Maintenance data with respect to costs and application times are usually recorded in a collective bridge log, such as the *Bauwerksbuch* according to the German DIN 1076. Alternatively, information on maintenance costs may be obtained from the accountant department of the bridge authority. In order to create a data basis for all maintenance activities, the costs for each activity as they are found in the maintenance records, must be transformed to a common reference year. Such a transformation, for example to the year of bridge completion, can be achieved with reference to the construction price index of the German *Statistisches Bundesamt* (2011) by applying the equation 4.60, which is introduced in the below sub-section 4.7.4.

## 4.7 Performance Indicators

### 4.7.1 Overview

Performance indicators for maintenance strategy assessment and evaluation condense the long-term bridge performance—relating to *condition*, *structural safety*, *economy*, *society & culture*, and *environment*—to single quantitative values. This becomes necessary because strategic bridge maintenance planning refers to a long time horizon and thus strategies must also be assessed on this basis. The condensation is achieved by averaging the continuous performance developments but also—in the case of structural safety—by using the resulting service life as the single-value indicator. Moreover performance indicators should be expressed in appropriate units in order to avoid distortive monetarisation. For example, maintenance costs should be expressed in monetary values while the traffic flow performance should be expressed in terms of percentage of the design traffic capacity.

In the following, the present section introduces a broad range of indicators. While the indicators that refer to safety, bridge condition, and economy, are based on the relevant literature, the indicators describing societal and cultural aspects must be newly developed. This is because, indicators for these latter two aspects are only exceptionally found in the literature and, if so, they are usually described in monetary terms as equivalent users' costs. Such monetarisation methods are also often applied to the environmental issues.

### 4.7.2 Structural Safety

For the assessment of the structural safety, the SAMS program applies the *deterministic code-based approach*, which is described in sub-section 2.7.3. Accordingly, the overall structural safety is described by the indicator $\eta$:

$$\eta = \min_i \left( \eta_{Elem,i} \right) \tag{4.53}$$

with

$\eta_{Elem,i}$ degree of fulfilment of the i$^{th}$ element (both bending and shear)

Also, the indicator $CI_\eta$ is defined in order to include the level of structural safety in the overall condition assessment by the below described indicator $CI_{bridge}$: Bridges with $\eta \geq 1$ are uniformly rated with $CI_\eta=1$ while bridges with $\eta<1$ are uniformly rated with $CI_\eta=0$.

### 4.7.3 Condition

At the element level, the present thesis considers a condition indicator, which represents the bridge equipment and concrete surface condition on the basis of a worst case classification scheme, as it is described by equation 4.54. On a global level, two types of rating schemes are applied for aggregating the element ratings to an overall performance: The first one is also of the worst case classification type, which is expressed in terms of the indicators $CI_{min}$ (for example $CI_{cs,min}$ or $CI_{A,min}$) according to equation 4.55. The second one, which is specified in terms of equation 4.56, describes the resulting bridge condition in terms of a weighted average of all SAMS-elements and refers to the concrete surface ($CI_{cs,res}$), the asphalt pavement ($CI_{A,res}$), the bearings ($CI_{Bear,res}$), the expansion joints($CI_{Exp,res}$), and finally the drainage system ($CI_{Drain,res}$). Also a resulting bridge condition $CI_{bridge}$ is introduced in the form of equation 4.57, which is defined as a minimum value superposition of all the above mentioned condition indicators and the $CI_\eta$-indicator for the structural safety. All these indicators are interim variables, which are not used for strategy assessment itself but serve as input values for the actually finally considered performance indicators. This is because each one of these indicators expresses the bridge condition only at different points in time, but does not give comprehensive information from a service life perspective.

For the purpose of strategy assessment with respect to the performance aspect *condition*, the present thesis introduces the condition indicator $CI_{bridge,av}$, which describes the average bridge condition over the whole service life (equation 4.58) with reference to the interim condition indicator $CI_{bridge}$. It shall also be mentioned, that the above described interim condition indicators, are also used as a basis from which to calculate further indicators from other performance fields, specifically from *society & culture*. An example for this is described in the below sub-section 4.7.5, which introduces the indicator *availability* $a(t)$.

$$CI_{\{Name\},i} = \min_k \left( CI_{\{Name\},k,i} \right) \tag{4.54}$$

$$CI_{\{Name\},\min} = \min_i \left( CI_{\{Name\},i} \right) \tag{4.55}$$

$$CI_{\{Name\},res} = \sum_{i=1}^{N_{Elem}} w_{\{Name\},i} \times CI_{\{Name\},i} \tag{4.56}$$

$$CI_{bridge} = \min\left( CI_{cs,res}, CI_{A,res}, CI_{Exp,res}, CI_{Bear,res}, CI_{Drain,res}, CI_{\eta,res} \right) \tag{4.57}$$

$$CI_{bridge,av} = \frac{1}{T_{SL}} \int_0^{T_{SL}} CI_{bridge}(t)\mathrm{d}t \tag{4.58}$$

with

| | |
|---|---|
| $CI_{\{Name\},i}$ | condition index of the i$^{th}$ element with respect to the component {Name} |
| $\{Name\}$ | name of structural component: *CS* (concrete surface), *A* (asphalt pavement), *Exp* (expansion joints), *Drain* (drainage system), *Bear* (bearings) |
| $k$ | index that is necessary in the case that the component's rating is calculated from sub-components' ratings. This, for example, is the case for the concrete surface, where $CI_{\{Name\},i} = \min\left( CI_{\{Name\},web,i}, CI_{\{Name\},bottomslab,i}, \ldots \right)$. |
| $N_{Elem}$ | number of elements (relevant cross sections) |
| $w_{\{Name\},i}$ | weighting factor of bridge element '*i*' $\left( \sum w_{\{Name\},i} = 1 \right)$, see sub-section 4.3.4 |
| $CI_{\eta}$ | binary indicator of the structural safety (see sub-section 4.7.2): $CI_{\eta} = 1$ if $\eta \geq 1$ and $CI_{\eta} = 0$ if $\eta < 1$ |

### 4.7.4 Economic Aspects

The basic methodologies for assessing the economic performance in terms of the monetary indicator *net present value of the total owner costs* ($C_{0,owner}$) are introduced in the above sub-section 2.7.4. This indicator $C_{0,owner}$ is calculated according to the above equation 2.9, which includes all contributions from bridge design and construction to inspection maintenance and decommissioning. As it is discussed in sub-section 2.7.5, the present thesis applies a strategy assessment approach where exclusively the owner costs are treated in monetary terms while all other aspects—for example the so-called user or environmental costs in the form of delays, traffic accidents, and $CO_2$ emissions—are not considered to belong to the economic domain. These are represented by non-monetary performance indicators in a direct and undistorted way, for example as societal and cultural performance indicators as in sub-section 4.7.5.

Crucially, when using $C_{0,owner}$ for comparative strategy evaluation, an important point must be taken into account: Alternative strategies usually yield different results for the service life $T_{SL}$. The effect of such a different service life is that bridges with a longer service life can lead to higher net present value of maintenance costs compared to bridges with a shorter service life, simply because more maintenance activities are conducted over the years. But, obviously, a comparison of these costs disregards the benefits of the longer service and the disadvantages of earlier replacement of the short living bridge.

One solution to this problem would be to assess all strategies over a predefined period, for example a 100 year time-horizon. Such an approach is applied, for instance, by Neves and Frangopol (2005). Also the German RI-WI-BRÜ applies this approach and in addition considers the residual value for bridges at the end of the defined assessment period; hereby the RI-WI-BRÜ-approach counteracts potential distortions in the net present value that may arise in the case if major repairs are applied shortly before the end of the assessment periods.

An alternative solution to this problem is to assume a theoretically infinitely repeating cycle of bridge replacement and maintenance of follow-up structures (Fig. 4.22 (a)), as it is for example applied by Pommerening et al. (2008: 44–45). By this approach, the eternal replacement-maintenance cycle is expressed by the *follow-up costs* $C_f$ that are calculated by discounting the eternal cash-flow to t=$T_{SL}$. By following this approach, differences in the service life have the following monetary effect: The shorter the service life $T_{SL}$ is, the higher is the contribution of the follow-up costs ($C_f$) to the net present value of the owner costs ($C_{0,owner}$) because of the shorter discounting period. Consequently, the above described problem is counteracted by penalising shorter service lives with higher costs.

In principle, the present thesis decides for the latter approach with the detailed consideration of the strategy-specific implications of the service life: The follow-up costs $C_f$ are calculated in the scope of the present thesis in analogy to the method applied by Pommerening et al. (2008: 44–45) as the net present value of an infinite series of cash flows that results from repeated cycles of bridge replacement and in-service maintenance. However, in contrast to these authors the present thesis follows an approach where the follow-up costs $C_f$ are calculated by assuming an infinite repetition of replacement and of the actually underlying maintenance strategy (Fig. 4.22 (b)); it is not done by assuming a uniform follow-up strategy (Fig. 4.22 (a)) as by Pommerening et al. (2008: 44). This assumption is made because then the follow-up costs $C_f$ represent the individual maintenance strategy, while according to Pommerening et al. $C_f$ is strategy-independent. As a result, the performance indicator $C_{0,owner}$, as it is defined in this thesis, is even more representative for individual strategies.

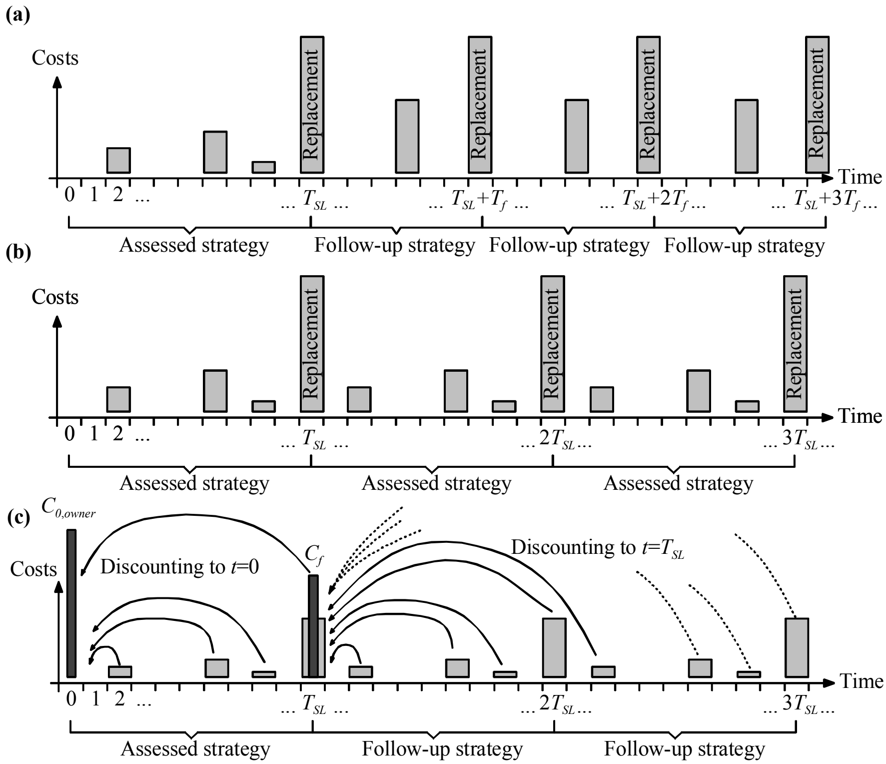



**Fig. 4.22:** Cash flow series for the calculation of the follow-up costs $C_f$: (a) Approach according to Pommerening et al. (2008: 44–45) with a uniform follow-up strategy; (b) approach with an infinite repetition of the assessed strategy; (c) discounting of strategy cash flow to $t$=0 and discounting of follow-up cash flow to $t$=$T_{SL}$ in order to calculate $C_f$, which again is discounted to $t$=0. (Note: The style of (a) and (b) is based on *Bild 18* and *Bild 19* in Pommerening et al. 2008).

By applying this approach, the net present value of the total owner costs ($C_{0,owner}$) is calculated according to the following equation 4.59. There, the initial costs for bridge design and construction ($C_{d+c}$) are assumed to be the same for all maintenance strategies, both in terms of monetary value and distribution over time, and thus are not considered for the calculation of the total owner costs ($C_{0,owner}$). In contrast to this, the costs $C_{rep}$ for bridge replacement—demolition as well as design and construction of the new bridge—are individually taken into account for estimating the follow-up costs, which is simply for the reason that they occur at the strategy-specific end of the service life $T_{SL}$.

$$C_{0,owner} = \left(1 + \frac{1}{(1+i)^{T_{SL}} - 1}\right) \sum C_{maint,j} (1+i)^{-T_{app,j}} + \frac{C_{rep}}{(1+i)^{T_{SL}} - 1} \tag{4.59}$$

This follows from substituting $C_{cycl} = C_{rep} + \sum C_{maint,j} (1+i)^{-T_{app,j}}$ in

$a_f = C_{cycl} \dfrac{i(1+i)^{T_{SL}}}{(1+i)^{T_{SL}} - 1}$ and finally substituting $C_f = \dfrac{a_f}{i}$ in

$$C_{0,owner} = \sum C_{maint,j} (1+i)^{-T_{app,j}} + C_f (1+i)^{-T_{SL}}$$

with

| | |
|---|---|
| $C_{maint,j}$ | maintenance costs for the j[th] activity |
| $T_{app,j}$ | application time of the j[th] activity |
| $C_f$ | follow-up costs for infinite cycles of replacement and strategy repetition |
| $C_{rep}$ | costs for replacement (demolition, design, construction) of the bridge |
| $a_f$ | equivalent annuity of the follow-up costs |
| $C_{cycl}$ | NPV of the costs for a cycle of replacement and strategy repetition |

In order to account for the time dependent impact of costs, the input data for each activity ($C_{maint,j}$) should refer to a common time base. In the scope of the present thesis, this is accounted for by transforming the price information, which is taken either from literature or from case study data, to the year of bridge completion. The actual transformation is achieved with reference to the construction price index of the German Statistisches Bundesamt (2011: Table 5):

$$C_{maint,j} = C_{maint,j,T_{data\ source}} \frac{CPI(T_{ref})}{CPI(T_{data\ source})} \tag{4.60}$$

with

| | |
|---|---|
| $C_{maint,j,T_{data\ source}}$ | maintenance costs for the j[th] activity according to the data source (e.g. literature or case study) with time basis $T_{data\ source}$ |
| $T_{ref}$ | refernce year for calculation of the NPV, e.g. year of bridge completion |
| $T_{data\ source}$ | time basis of the cost data (e.g. literature or case study) |
| $CPI(t)$ | construction Price Index at time $t$ according to Statistisches Bundesamt (2011: Table 5) |

### 4.7.5 Functional as well as Societal and Cultural Aspects

The present thesis considers three performance aspects from the societal and cultural domain for the assessment and evaluation of maintenance strategies. Of these, the aspect *traffic flow performance* also constitutes the central functional aspect of a bridge in its role as a traffic link.

- service life (preservation)
- traffic flow performance
- aesthetic appearance

**Service Life (preservation)**

The performance indicator *service life* ($T_{SL}$) is used in the sense of the *technical service life* which is defined by the '*time in service until a defined unacceptable state is reached*' (fib 2002: 5). Such an unacceptable state occurs when one or more performance requirements are violated and no maintenance activities—except for complete replacement—can be possibly applied in order to restitute a satisfying bridge performance. Other than purely technical reasons for not applying necessary maintenance measures may be that the structure is functionally obsolete due to changes in requirements (*functional service life* according to fib 2002: 5), or that replacement is economically more advantageous than keeping the existing bridge in service (*economic service life* according to fib 2002: 5).

However, the functional service life is not considered for strategy assessment in the scope of the present thesis. Generally, such a service life definition may be quite easily included in the SAMS-analysis by defining time-depending performance thresholds within the strategy program; nevertheless, such approaches are not applied during the present investigation. Also economic analyses with respect to the *'replace or maintain'-decision* are not explicitly conducted within the individual strategy assessment procedures; however, such analyses can be—to some extent—implicitly included in a comparison of replacement-strategies to (proactive) repair strategies.

In the SAMS program the service life limiting violation of performance requirements is implemented by the criterion $CI_{bridge}$=0, which uses the overall bridge condition indicator $CI_{bridge}$ (see equation 4.57). In the form of a *worst element classification index*, $CI_{bridge}$ reaches a value of zero in the case that any of the individual indices becomes zero and at the same time no maintenance activities can be possibly applied for curing this state.

**Traffic Flow Performance**

The traffic capacity of a bridge can fall back behind the initial capacity because of a deteriorating pavement condition, as a result of expansion joint defects, or due to traffic restrictions from maintenance works. The indicator *average availability* ($A_{av}$) is defined in the scope of the present thesis for capturing these effects. This indicator captures what percentage of the traffic flow, for which the bridge was originally designed for, can in average be provided for during the bridge service life. This average availability is derived from the variable *availability* ($a$), which describes the ratio of the actual traffic capacity to the initial traffic capacity at each point in time (equation 4.61). The indicator $A_{av}$ is defined—according to the mean value theorem for integration—as the mean value of the continuous function $a(t)$ over the service life, and is described by equation 4.62.

$$a(t) = \frac{\text{traffic capacity } (t)}{\text{initial traffic capacity}} \tag{4.61}$$

$$A_{av} = \frac{1}{T_{SL}} \int_0^{T_{SL}} a(t)\mathrm{d}t \tag{4.62}$$

The *traffic capacity* of a bridge is defined in the scope of the present thesis as the maximum traffic flow $q_{max}$ that the bridge allows for. This maximum traffic flow can be estimated on the basis of a simple linear traffic flow model which has been proposed by Greenshields in the early 30's of the 20$^{th}$ century (Greenshields 1934, 1935). His model dominated traffic flow theory for over 50 years (FHWA 2001: 2-19) and many investigations showed good correlations with field data (Gerlough and Huber 1975: 49); however, subsequent investigations revealed some shortcomings and lead to the formulation of more sophisticated models (FHWA 2001: 2-19). Nevertheless, in the scope of the present thesis Greenshilds' classic approach is applied because of its simplicity that allows—after several transformations as it will be demonstrated in the following—for a simple expression of the availability as a function of the pavement and expansion joint condition.

Greenshields describes a simple linear relationship between the mean travel speed $v_m$ and the traffic density $k$ in the form of equation 4.63 with two road specific parameters. The first is the mean free travel speed ($v_{m,f}$), which occurs when the traffic density is zero so that no interference between the vehicles occurs and each vehicle can drive freely. The second parameter $k_j$ stands for the traffic jam density at which the travel speed becomes zero. The traffic flow $q$ then can be expressed as the product of the mean travel speed $v_m$ and the traffic density $k$ (equation 4.64).

$$v_m = v_{m,f}\left(1 - \frac{k}{k_j}\right) \tag{4.63}$$

$$q = v_m\, k \tag{4.64}$$

with

| | |
|---|---|
| $v_m, v_{m,f}$ | mean travel speed ($m$) and mean free ($m, f$) travel speed with no interference |
| $k, k_j$ | traffic density, traffic jam ($j$) density ($v_m$=0) |

It can be shown by applying basic calculus that the traffic flow $q$ reaches a maximum value for a mean travel speed of $v_m$=0.5 $v_{m,f}$ so that the maximum traffic flow, and thus the traffic capacity, can be expressed by equation 4.65. Consequently, the availability can be expressed in the form of equation 4.66 as the ratio of the actual mean travel speed $v_m$ to the initial mean travel speed $v_{m,0}$. This is found for the simplifying assumption that only the mean travel speed $v_m$ is affected by the road condition, while the traffic jam density $k_j$ remains unchanged. This assumption is justified because, according to Stütze (2004: 16, 64–66), traffic jams usually do not follow from a deteriorated road condition but from capacity restrictions such as traffic lane restrictions, while the road condition significantly influences the travel speed.

$$q_{\max} = 0.25\, v_{m,f} k_j \tag{4.65}$$

$$a(t) = \frac{\text{traffic capacity } (t)}{\text{design traffic capacity}} = \frac{q_{\max}(t)}{q_{\max,0}} = \frac{v_{m,f}(t)}{v_{m,f,0}} \quad \left(\text{with } k_j = \text{const.}\right) \tag{4.66}$$

with

| | |
|---|---|
| $q_{\max}$ | maximum traffic flow |
| $v_{m,f,0}$ | initial mean free travel speed (new carriageway) |
| $q_{\max,0}$ | initial maximum traffic flow (new carriageway) |

In order to calculate the availability and its average value over the service life, the mean travel speed $v_m$ can be defined as a function of the pavement ($CI_A$, see 4.3.4) and expansion joint condition ($CI_{Exp}$, see 4.3.4). In the scope of the present thesis, a simple model is formulated, which refers to the work of Stütze (2004: 64–66), who describes the relationship between mean travel speed $v_m$ and the German roughness index $AUN$, which rates the road condition in terms of surface irregularities in the driving direction (Fig. 4.23 (a)).

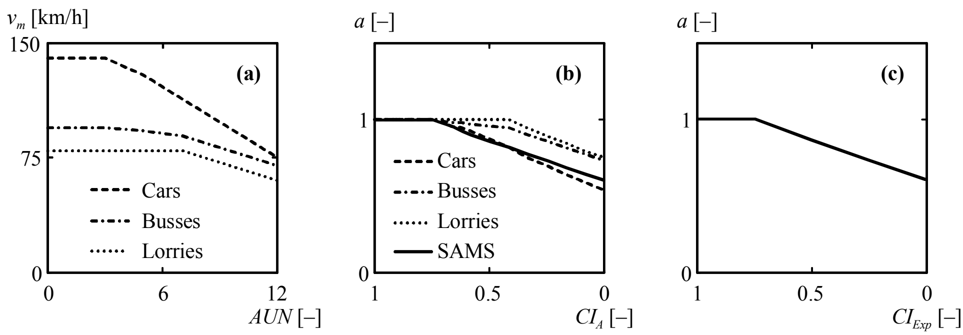



**Fig. 4.23:** Development of the availability-condition relationships for the pavement and the expansion joints: (a) Relationship between mean travel speed and road condition according to Stütze (2004: 64–66); (b) transformation of the data provided by Stütze (2004: 64–66) to an $a$-$CI_A$ relationship and definition of the curve that is implemented into SAMS; (c) analogous $a$-$CI_{Exp}$ curve.

The curves describing these $v_m$=f($AUN$) relationships are transformed to a=f($CI_A$) curves, where the indicator $CI_A$ represents the condition of the asphalt pavement (see see 4.3.4). This is done by applying a simple linear transformation ($CI_A$=1–$AUN$/12) of the 1 (excellent) to 12 (very poor) $AUN$ value range in Fig. 4.23 (a) to the 1–0 scale that describes the condition of the asphalt pavement (Fig. 4.23 (b)). For simplifying reasons, only one $a$=f($CI_A$) curve, which lies between the curves for cars on the one side and busses and lorries on the other side, is applied in the SAMS program. Also, an analogous relationship between the traffic availability and the condition of the expansions joints is assumed (Fig. 4.23 (c)). Both relationships are described by the following equation 4.67.

$$a\left(CI_{A/Exp}\right)=\begin{cases}1 & \text{for } 1.00 \geq CI_{A/Exp} \geq 0.75\\ 1\text{-}0.4/0.75\times\left(0.75\text{-}CI_{A/Exp}\right) & \text{for } 0.75 > CI_{A/Exp} \geq 0.00\end{cases} \tag{4.67}$$

A further availability reduction can occur because of maintenance activities. For example a closure of one lane leads to a reduction of availability by 50% in case the carriageway on the bridge consists of two lanes. This effect is taken into account by the factor $\nu_{traf}$. The resulting availability, which takes into account all described effects is given by equation 4.68.

$$a\left(Con_A, Con_{Exp}, \delta_{maint}\right)=\nu_{traf}\times\min\left[a\left(Con_A\right), a\left(Con_{Exp}\right)\right] \tag{4.68}$$

The following Fig. 4.24 shows an example calculation of the average availability for hypothetical developments of the asphalt pavement and the expansion joint condition (Fig. 4.24 (a) and (b)). The average availability is calculated as $1/T_{SL}$ times the area under the resulting availability curve in Fig. 4.24 (c). The availability curve follows from the traffic effects of $CI_A$ and $CI_{Exp}$ according to equation 4.67 and by superposition according to 4.68 including an additional 50% capacity restriction during the maintenance works.

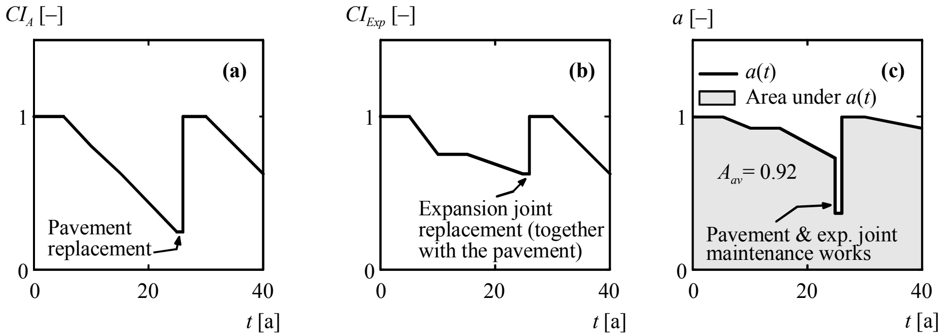



**Fig. 4.24:** Example calculation of the average availability $A_{av}$: (a) Hypothetical condition development of the asphalt pavement $CI_A$; (b) hypothetical condition development of the expansion joints $CI_{Exp}$; (c) resulting availability curve $a(t)$ and average availability $A_{av}$

**Aesthetic Appearance**

Strategy assessment with respect to aesthetic aspects is implemented in the SAMS program with the help of the indicator *relative time of unsatisfactory appearance* ($RT_{ua}$). Being solely applied for maintenance strategy assessment, the indicator $RT_{ua}$ does not, however, refer to the aesthetic quality in terms of structural form or other design aspects. Instead, this indicator exclusively refers to the progressing decay of the concrete surface, whose condition significantly contributes to the overall appearance of the bridge. The indicator *relative time of unsatisfactory appearance* ($RT_{ua}$) has been introduced by Dette and Sigrist (2010, 2011) and is defined as the fraction of the service lifetime in which the condition of the concrete surface, which is expressed in terms of the indicator $CI_{CS}$ (see 4.3.4), is below a specifically defined aesthetic threshold:

$$RT_{ua} = \frac{1}{T_{SL}} \sum \mathrm{d}T_{ua} \tag{4.69}$$

with

$dT_{ua}$ time intervall where $CI_{cs,res}$ is below the threshold $CI_{cs,ua}$

$CI_{cs,ua}$ threshold of unsatisfactory appearance

Self-evidently, the aesthetic threshold $CI_{cs,ua}$ lies above the critical damage level at which major interventions become necessary due to severe deterioration of the concrete surface, which causes the overall bridge condition $CI_{bridge}$ to become zero. For instance, a limited amount of local spalling of the concrete cover without significant corrosion of the reinforcement is not classified as a critical damage; however, it might be regarded as unsatisfactory in aesthetic terms. Unfortunately, the actual definition of the aesthetic threshold in terms of damage extent and its quantification by an explicit value between 0 and 1 is not a self-evident choice but rather a matter of personal judgement. In the scope of the present thesis, the threshold value $CI_{cs,ua}$ is set to a level of 0.75 for the average condition of the concrete surface of the whole bridge $CI_{cs,res}$. By definition of the concrete surface condition rating scale in Appendix II, such a value of $CI_{cs,res}$ describes a state where, on average, the whole structure shows mostly minor defects to such an extent that a minor adverse effect on the aesthetic appearance occurs. The following Fig. 4.25 illustrates the calculation of the performance indicator $RT_{UA}$ for an example development of the indicator $CI_{cs,res}$ over a reference time span of 40 years..

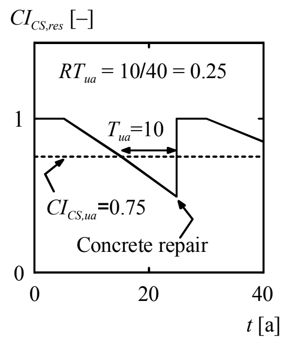



**Fig. 4.25:** Example calculation of $RT_{ua}$

### 4.7.6 Environmental Performance Aspects

Performance indicators for environmental strategy assessment are not defined in the scope of the present thesis. However, for further investigations such indicators can also be easily defined, for example in the form of *$CO_2$ equivalent emissions, primary energy consumption*, and *landfill production*. Such indicators may both account for the environmental impact of individual maintenance activities and for the impact resulting from the time dependent traffic flow performance of the bridge. The first indicator type can encompass impacts such as landfill production from removal of the old asphalt pavement as well as the $CO_2$ emissions and energy consumption in connection with the construction of the new pavement. The second environmental impact type results for example from increased fuel consumption due to detour and congested traffic.

## 4.8 Maintenance Strategies

The SAMS program implements maintenance strategies in the form of sub-programs within the overall program routine. These strategy sub-programs control the program run by selecting and scheduling maintenance activities according to specific criteria and decision rules. In this way each strategy sub-program constitutes the *underlying concept of the object-oriented, holistic and long-term-oriented selection, combination and application of maintenance measures*, which is the definition for the term maintenance strategy in the scope of this thesis (see sub-section 2.6.2). These strategies can be categorised with respect to their underlying concepts into strategy types such as *reactive repair* or *preventive-proactive* (Table 2.3). In detail, each maintenance strategy sub-program within the SAMS program-package is specified in terms of *maintenance application rules* which typically include the following:

- *Specific maintenance application criteria*: These criteria are defined in the form of threshold values for specific performance indicators and associated maintenance activities, which are applied in the case of threshold violation. For example, such a criterion can be defined as the combination of a threshold value of 0.50 for the overall pavement condition $CI_{A,res}$ and the activity *pavement resurfacing* which is applied if the indicator $CI_{A,res}$ falls below a value of 0.5.
- *Combination rules*: These rules define pairs or sets of activities, which are always applied in combination—for example the replacement of the expansion joints together with bridge deck reconstruction or pavement replacement.
- *Time based maintenance plans*: These plans describe action plans for the routine application of selected activities at certain intervals or pre-defined points in time. An example for this is a routine application of silane treatment in 10 year-cycles.

## 4.9 Strategy Assessment and Evaluation

### Strategy Assessment

For strategy assessment, the SAMS program provides the results of the stochastic simulation in the form of value distributions for performance indicators. These value distributions describe the outcomes of the repeated calculation of the assessment indicators from a stochastic sample of input parameters, which describe the deterioration processes and the maintenance activities.

In the scope of the present thesis, the SAMS program considers functional, economic and societal as well as cultural performance aspects in terms of indicators that describe the owner's costs ($C_{0,owner}$), the service life ($T_{SL}$), the traffic flow performance ($A_{av}$), and the aesthetic appearance ($RT_{ua}$). Whereas, the structural safety ($\eta$) as well as environmental aspects are not explicitly applied for strategy assessment.

Neglecting structural safety for strategy assessment and evaluation is justified by considering that a safety level which exceeds the code requirements may be comfortable but usually has not much direct gain. Nevertheless, the structural safety is implicitly included in the calculation of the overall bridge condition and the service life. Also the environmental impact is not considered for service-life strategy assessment—the reason for this is to limit the scope of the simulation study of the present thesis.

The below Fig. 4.26 exemplifies how the SAMS program provides the strategy assessment results in the form of value distributions for the performance indicators.

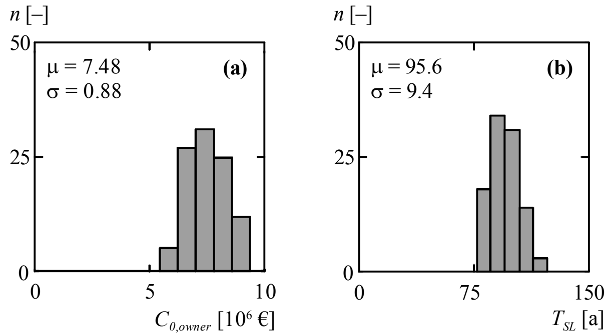



**Fig. 4.26:** Example value distributions of performance indicators for strategy assessment based on a stochastic sample of input parameter values of size $N$=100 (a) net present value of the owners cost; (b) service life. (*Note*: μ stands for sample mean value and σ for sample standard deviation)

**Strategy Evaluation**

It must be kept in mind that the multi-objective strategy evaluation process is not of strictly analytical nature but rather exhibits interpretative facets, as it is discussed in section 2.8. Bearing this in mind, the SAMS program only goes as far as to provide so called *strategy performance profiles* (see sub-section 2.7.1), while the final strategy evaluation process is still left to interpretative personal judgement.

These strategy performance profiles depict the results for selected performance indicators in the form of star plot diagrams. They use a uniform scale from the lowest value 1 to the highest value 5 to which all resulting values are transformed by the following linear transformation function (equation 4.70).

$$u_{\{Name\},k} = 1 + \frac{4}{P_{\{Name\},best} - P_{\{Name\},worst}} \left( P_{\{Name\},k} - P_{\{Name\},worst} \right) \quad (4.70)$$

with

| | |
|---|---|
| $u_{\{Name\},k}$ | value of the performance indicator {*Name*} (e.g. $CI_{cs}$ or $T_{SL}$) on a 1 to 5 scale as it occurs for strategy *k* |
| $P_{\{Name\},best}$ | best single performance value that occurs for all strategies with respect to the indicator {*Name*} (e.g. minimum value for $C_{0,owner}$) |
| $P_{\{Name\},worst}$ | worst single performance value for all strategies |
| $P_{\{Name\},k}$ | actual performance value for strategy *k* |

In addition to these graphic strategy-profiles for qualitative-intuitive strategy evaluation, the SAMS program also applies a non-monetary transformation function, which calculates a single-value performance-indicator. This transformation function is of the additive type as in equation 2.14 and it applies a linear transformation function for all individual performance aspects as described in Fig. 2.10 (c). This linear and non-monetary transformation function is defined by the following equation 4.71 with the weighting factors ($w$) for transformation being derived from the preference profile according to equation 4.72. However, it must be kept in mind that results found for this indicator must always be interpreted together with the above described strategy performance profiles. This is because otherwise unwanted overcompensation effects of one performance aspect at the expense of another aspect can be concealed within the single-value performance indicator, as it is illustrated by the example evaluation in sub-section 2.8.5.

$$u_{res,k}\left(u_{\{Name\,1\},k}, u_{\{Name\,2\},k}, \ldots, u_{\{Name\,n\},k}\right) = \sum_{1}^{n} w_i u_{\{Name\,i\},k} \tag{4.71}$$

$$w_i = \frac{R_i}{\sum_j R_j} \tag{4.72}$$

with

| | |
|---|---|
| $u_{res,k}$ | resulting performance for strategy $k$ |
| $u_{\{Name\,i\},k}$ | value of the performance indicator $\{Name\,i\}$, e.g. $u_{C_{0,owner},Strategy\ S1}$ |
| $w_i$ | weighting factor for performance indicator $i$ |
| $R_i$ | preference rating for performance aspect $i$ |

The following Fig. 4.27 exemplifies the transformation of the assessment results for three different strategies as well as the presentation of the resulting performance profiles in the form of star plot diagrams. Fig. 4.27 (a) depicts the transformation of the performance indicator values according to the above equation 4.70 to the value $u$, which is defined in the range from 1 to 5. For example, a service life of 86 years is transformed to $u$-value of 2.5 (Fig. 4.27 (a)). The three star plot diagrams in Fig. 4.27 (b) show the performance profiles that result for three different strategies $S_1$, $S_2$, and $S_3$ from the same sample element, which is a specific combination of deterioration and maintenance input parameter values. Finally, the three star plot diagrams in Fig. 4.27 (c) exemplify the performance profiles as they result from a sample of size $N$=25 together with the profiles that represent the sample mean performance values.

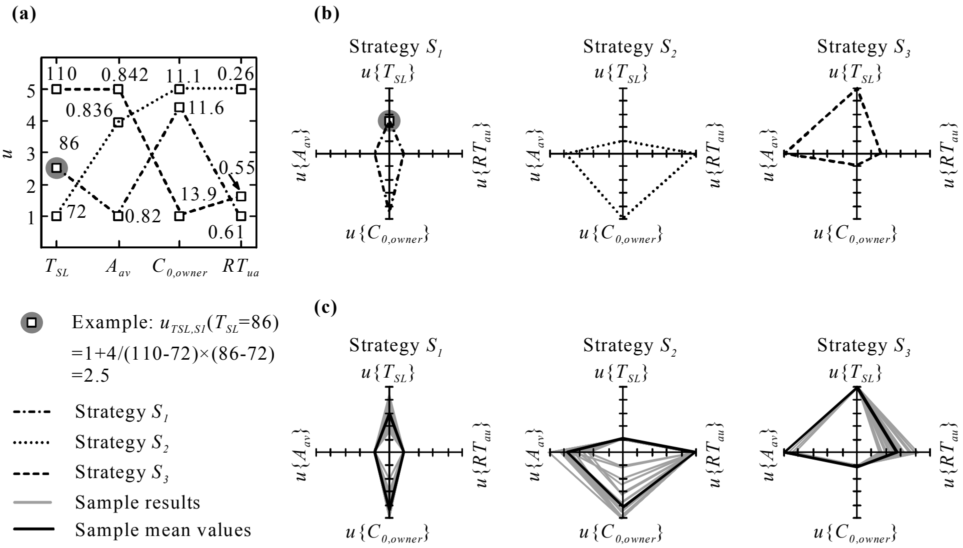



**Fig. 4.27:** Example: (a) transformation of performance indicator values to a 1–5 scale; (b) performance profiles for the sample element that is transformed in (a); (c) performance profiles for 25 sample elements (grey lines) and for the sample mean values (black lines).

## 4.10 SAMS Program Structure

As the first step the input data are specified by the user of the program (**{1}** in Fig. 4.28 (a)). These input data provide the structural representation of the bridge as well as the detailed description of the deterioration mechanisms and the maintenance activities. Also the strategy-independent minimum performance requirements, which must be fulfilled in any case, are defined in this step. Such minimum requirements appear for example in the form of $\eta \geq 1.0$ and $CI_{cs,res} \geq 0$.The structural representation, as it is described in section 4.3 of the present thesis, is achieved in the form of detailed data regarding structural dimensions, material properties, condition of equipment components and concrete surfaces. The relevant internal forces in each representative bridge element must be calculated beforehand and are provided to the SAMS program in the form of explicit values.

The deterioration mechanisms are specified in the form of parameters, which describe the deterioration rates and initiation times ($\alpha_1$ and $T_i$) in the form of stochastic distributions. The deterioration levels $\delta_k$, where the deterioration rate changes, is used as a deterministic input parameter. All the stochastic input parameters are described for three different deterioration intensity levels—high (H), medium (M), and low (L)—in order to account for varying circumstances with respect to vulnerability and exposition.

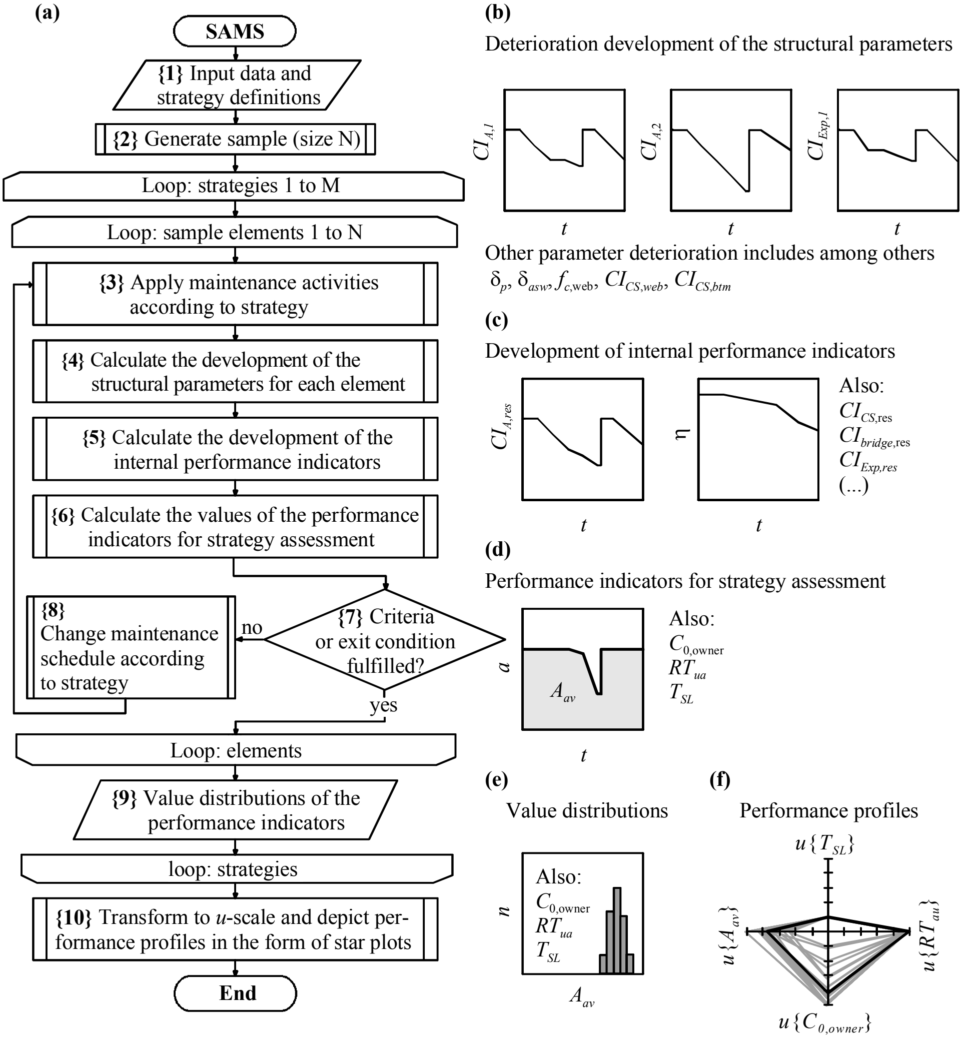



**Fig. 4.28:** *SAMS* program: (a) flowchart; (b) time dependent development of structural parameters as a result of the deterioration processes and the maintenance effects (see step {4} in flowchart); (c) time dependent development of internal performance indicators, which are used as interim variables (see step {5} in flowchart); (d) performance indicators for strategy assessment (step {6} in flowchart); (e) value distributions that result from the repeated calculation of the performance indicators for the sample size N (see step {9} in flowchart); (f) depiction of the resulting strategy performance profiles in the form of star plot diagrams (see step {10} in flowchart). (Note: This figure is based on Fig. 1 in Dette and Sigrist 2010.)

The maintenance parameters, which describe the effects on the deterioration mechanisms, are also defined in the form of stochastic distributions for three intensity levels ($\nu_1$, $\nu_2$, $dt_1$, and $dt_1$) or as deterministic parameters ($\Delta\delta$, $\delta_n$,). Additional parameters represent the stochastically distributed costs $C_{app}$ and the deterministic effects on the traffic flow ($\nu_{traf}$ and $dt_{traf}$).

As it is described in section 4.6 of the present thesis, the input parameters should, whenever possible, be adjusted to field data, such as bridge inspection reports, in order to provide more reliable results.

Also, this first step encompasses the definition of maintenance strategies in the form of sub-programs that control the overall program run by selecting and scheduling maintenance activities according to strategy specific criteria and decision rules.

After this, a subroutine generates in step **{2}** a stochastic sample, which contains value combinations for the deterioration and maintenance parameters. This sample of a defined size *N* is generated by Monte Carlo simulation technique from triangular or beta distributions (see sub-section 4.4.1).

The actual strategy assessment and evaluation begins only after this sampling is completed. As it is described in section 4.9, the strategy assessment includes calculating the value distributions of the performance indicators for each investigated strategy and after this generating the star plot diagrams for the strategy performance profiles. All this is achieved by iteratively applying the program steps **{3}** to **{10}** in the form of loops over the investigated strategies and the elements of the stochastic sample. The inner loop calculates the performance indicator values for each sample element, so that this loop over all *N* sample yields the value distributions for the performance indicators as they are described in section 4.9 and by Fig. 4.26. The outer loop calculates these value distributions for each investigated strategy. Eventually, the assessment results are transformed according to equation 4.70 and the strategy performance profiles are depicted in the form of star plot diagrams.

As a first step within the inner loop (step **{3}**), a preliminary maintenance schedule according to the actual strategy is generated. For this, the SAMS program uses strategy-functions which automatically select, combine and schedule maintenance activities on the basis of predefined decision-rules (see section 4.8). In most cases, the initial maintenance schedule contains no activities—except for strategies that apply a time-based maintenance regime. The effects of the different maintenance activities are superimposed according to the description in sub-section 4.5.1 by calculating the resulting reductions in the deterioration rates ($\nu$), as well as the increments or reset values ($\Delta\delta$, $\delta_n$). As will be seen later, this initial maintenance schedule is iteratively changed during the assessment process.

In the following step **{4}**, a sub-routine calculates the resulting deterioration development of the individual structural parameters, for example the pavement and expansion joint condition or the concrete strength at different SAMS-elements (see Fig. 4.28 (b)). This resulting deterioration development follows from the superposition of the maintenance effects with the natural deterioration development (see sub-section 4.5.1).

Based on this time-dependent deterioration of the structural parameters, the program calculates in step **{5}** internal performance indicators. These are interim variables, which are half way between the time-dependent development of the structural parameters and their condensation to performance indicators that represent the service life performance in a single numerical value. The SAMS program uses a large number of such interim indicators, including the concrete surface and the equipment condition (e.g. $CI_{cs,res}$, $CI_{A,res}$, $CI_{A,min}$) or the traffic availability $a(t)$.

In step **{6}** the performance indicators for strategy assessment ($T_{SL}$, $C_{0,owner}$, $RT_{UA}$, and $A_{av}$) are derived from the courses of the above described interim performance indicators. These strategy assessment indicators condense the performance of the structure over the whole service life.

Following this, an evaluation of the actual maintenance schedule, which has been defined in step {3}, is carried out: In step **{7}** it is checked if all criteria are fulfilled. These criteria refer to the above described minimum performance requirements, which are defined as input parameters in step {1}. They also can appear in the form of specific maintenance application criteria as performance thresholds whose violation triggers the application of certain maintenance activities. It is also checked if a specific exit condition is fulfilled. This exit condition is defined as a change in the violation of the same criterion of less than 1% from one loop run to the next—this is the case when the violation cannot be prevented by the applicable maintenance activities and therefore the end of service life is reached.

If one or more criteria are violated, the maintenance plan is changed according to the underlying strategy (step **{8}**) in order to prevent the violation. The procedure is repeated until all criteria are fulfilled or an exit condition applies, respectively.

After the performance indicators for strategy assessment ($T_{SL}$, $C_{0,owner}$, $RT_{UA}$, and $A_{av}$) have been calculated for all elements of the stochastic sample, the value distributions for the actual strategy are established in step **{9}**.

Finally, following the outer loop over the different strategies, the assessment results are transformed to $u$-scaled data with the help of equation 4.70 and depicted in the form a star plot diagram for each strategy, which displays the strategy performance profile (step **{10}**).

## 4.11 Conclusion

The present chapter introduces a new pragmatic approach to multi-objective maintenance planning, which accounts for the bridge specific context, especially with respect to specific requirements and the physical peculiarities. This approach includes three major phases: the development phase for the bridge specific requirements and maintenance objectives, the assessment phase with bridge specific modelling and simulation analysis, and the final evaluation of alternative maintenance strategies.

The core of this approach is the newly developed SAMS-program, whose name is the acronym for *simulation-based assessment of maintenance strategies*. The very detailed representation of the bridge structure, of the deterioration processes, of the maintenance activities, and of their manifold interactions enables the SAMS-program to account for bridge specific peculiarities to a great extent. This program produces resulting value distributions of performance indicators, which are used for strategy assessment: the net present value of the owner costs ($C_{0,owner}$), the service life ($T_{SL}$), the average availability ($A_{av}$), and finally the relative time of unsatisfactory appearance ($RT_{ua}$). With the help of these performance indicators a profound bridge specific and multi-aspect maintenance strategy assessment is conducted. Moreover, the program also interprets the assessment data in the form of star plot strategy profiles, which are used in the following evaluation phase.

The two other phases—the requirements / objective development phase and the strategy evaluation phase—apply tools and methods that have been introduced in the previous chapters: the questionnaire from the survey of bridge management practice (see sub-section 3.2.4) for identifying the bridge specific requirements and the method of qualitative profile-comparison for evaluating the maintenance strategies.

In conclusion, the newly developed approach allows for truly multi-objective and bridge specific maintenance planning.

# 5 Experimental Strategy Assessment and Evaluation

## 5.1 Introduction

This chapter carries forward the investigations of the previous chapters in the form of an experimental application of the newly developed approach by using the example of two bridges in Hamburg—the *Köhlbrandbrücke* and the *Brücke Bahrenfelder Chaussee*. In doing so, the present chapter demonstrates the practical applicability of the overall approach and specifically of the SAMS-program, which is used as the core instrument within the maintenance planning approach. First of all, the present chapter will explore if maintenance strategies in their outcomes are likely to show characteristic patterns of strengths and weaknesses with respect to different performance aspects (service life, costs, traffic, appearance) and if on these information a rational choice can be made for certain. Second, the present chapter introduces a heuristic method for developing maintenance strategies that are tailored to the bridge specific requirements.

## 5.2 Case Study Bridges

### Köhlbrandbrücke

The *Köhlbrandbrücke*, which was built in 1974, is briefly introduced in chapter 2 as one of the bridges within the survey on bridge management practice. There, the bridge is described as an arterial traffic link, which carries the main east-west traffic route with 2×2 lanes through the port of Hamburg and also connects the two motorways A7 and A1 (Gaffron et al. 2008). It crosses the *Köhlbrand*— a branch of the Elbe river—in the form of a 520 m long steel stay-cable bridge with a 325 m main span and also includes two gigantic prestressed-concrete approach ramps to the eastern and western side, which have an overall length of about 3200 m (Wittfoht et al. 1975: 133). The case study in the scope of the present thesis focuses on the prestressed concrete ramps of the bridge because Sigrist and Krahwinkel (2008: 169–174) have found as a result of a thorough expertise that these ramps are critical in a long term perspective due to progressing deterioration, while the steel bridge promises a relatively stable long term behaviour.

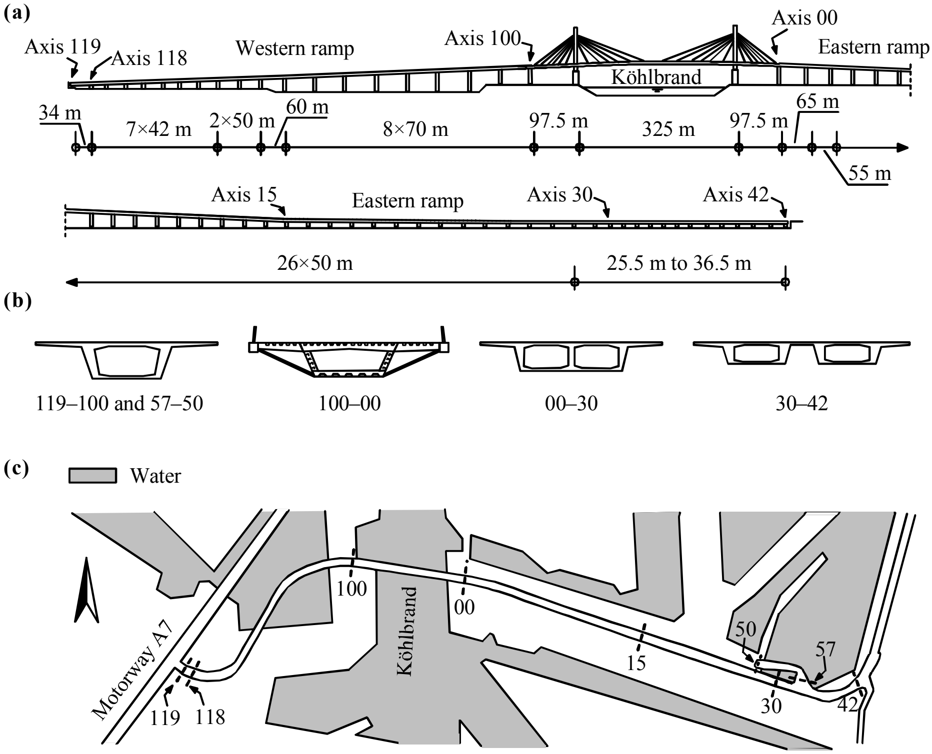



**Fig. 5.1:** Köhlbrandbrücke: (a) Elevation view of the whole bridge with span lengths and indication of the axes with expansion joints (119, 118, 100, 00, 15, 30, 42); (b) cross sections with indication (axes numbers) of the ranges where they are applied; (c) plan view of the bridge. (Note: This figure is based on *Bild 1*, *Bild 2*, and *Bild 3* in Wittfoht et al. 1975)

### Maintenance History of the Köhlbrandbrücke

In the following, only those maintenance activities that affect the ramps are considered because the actual example focuses on the prestressed concrete ramps. According to the maintenance log of the Köhlbrandbrücke (HPA 2006; HPA 2010) and inspection reports (Strom- und Hafenbau 1984, 1987a) the following major maintenance activities have been applied since the construction of the bridge in 1974:

- 1984: The bearings at axes 0 and 100 were repaired by replacing the broken rollers.
- 1986: Corroded tendons between axis 50 and 52 were conserved.
- 1987–1990: A major rehabilitation program was conducted along the whole length of the bridge. This program encompassed the repair of all expansion joints, major bridge deck reconstruction, and major kerb/sidewalk rehabilitation.

- 1993: Corroded tendons between axis 4 and 5 were conserved together with other tendons that were found to be exposed.
- 1997: The roller bearings, which had been located between axis 30 and 42, were replaced by elastomeric bearings.
- 1998–1999: All expansion joints were subject to repair works: The joints at axis 119, 118 and 15 to 57 in the year 1998 and the joints in axis 0 and 100 in the following year.
- 2002: The expansion joint in axis 100 was repaired again and also the kerbs/sidewalks of the western ramps were rehabilitated.
- 2004: All expansion joints—except for axis 0 and 100—were replaced.
- 2007: The exposed tendons between axis 00 and 01 were conserved.
- 2009–2012: A major rehabilitation program was conducted along the whole length of the bridge. It encompassed the following activities: tendon conservation as well as concrete repair of the box girder inside and outside (east 2010, west 2011), the rehabilitation of the eastern ramp's kerbs/sidewalks (2011), and also a complete replacement of the drainage pipes (2011).

**Brücke Bahrenfelder Chaussee**

Also the *Brücke Bahrenfelder Chaussee* is one of the bridges within the survey on bridge management practice in chapter 2. This bridge crosses the A7 motorway and links the Helmholtz Research Centre for particle acceleration—shortly known as the *Deutsches Elektronen Synchrotron (DESY)*—to the inner city districts of Hamburg via the main road Bahrenfelder Chaussee.

It is designed in the form of a continuous prestressed box-girder bridge with two spans of 28.01 and 32.68 m respectively. With its two parallel two-cell box-girder cross sections—one box-girder for each driving direction—it carries a deck with an overall width of 32.50 m. It was built in 1970 and is going to be replaced in 2016 by an enclosure of the A7 motorway (Hamburg 2010b/c).

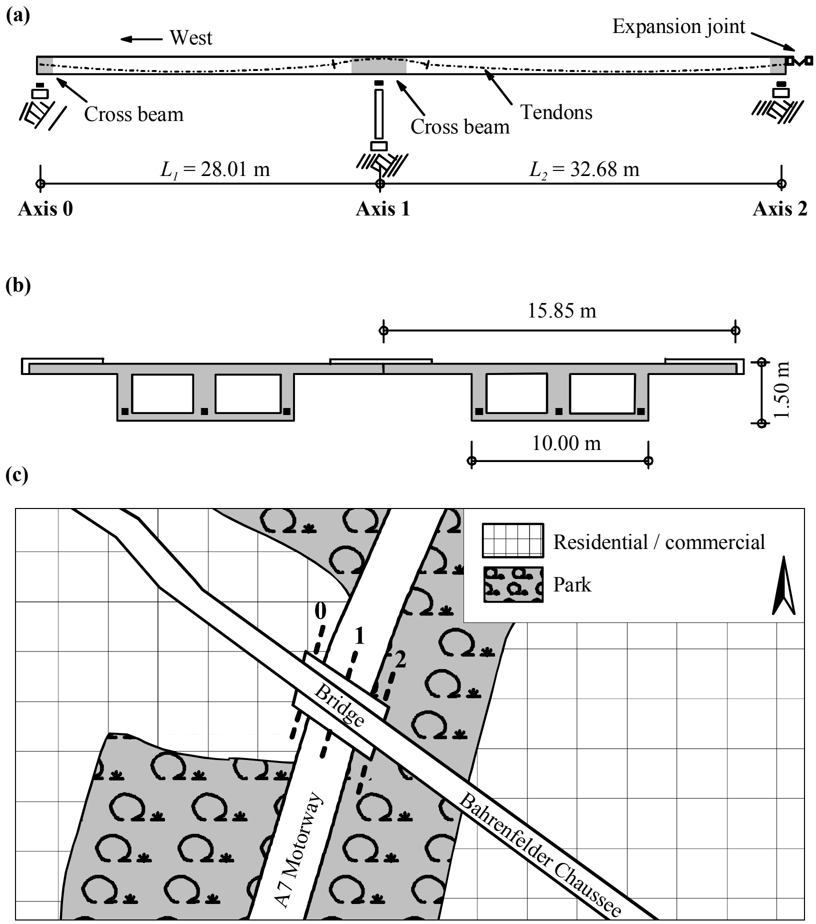



**Fig. 5.2:** Brücke Bahrenfelder Chaussee: (a) Elevation view of the whole bridge with span lengths and indication of the expansion joint location; (b) schematic cross section; (c) plan view of the bridge.

**Maintenance History of the Brücke Bahrenfelder Chaussee**

According to the maintenance log of the Brücke Bahrenfelder Chaussee (LSBG 2010) the following maintenance activities were applied during the 40-year period from 1970, which is the year of erection, to 2010:

- 1988: A major overhaul of the bridge was conducted. It included concrete repair and concrete coating as well as expansion joint replacement and asphalt resurfacing.
- 1995: The expansion joint was repaired and minor repair of the sidewalks concrete coating was conducted.
- 2005: Asphalt repair and resurfacing was applied.

## 5.3 Identification of Bridge Specific Preference Profiles

For both bridges, the bridge specific preferences have already been identified with the help of the items from the second to the fifth block of questions of the questionnaire from the survey (see sub-section 3.2.4). It appears that the profiles of the stakeholders' demands, of the owner's requirements, and of the maintenance objectives are consistent for each single bridge. Thus, for each bridge a characteristic preference profile for strategy evaluation can be identified. This profile is expressed in terms of *u*-values for the four performance indicators ($T_{SL}$, $C_{0,owner}$, $RT_{UA}$, and $A_{av}$), which represent the preferences with respect to service life, costs, aesthetic appearance, and traffic availability.

In the case of the Köhlbrandbrücke (Fig. 5.3), the profiles of the demands, the requirements, and the objectives actually coincide (the stakeholders' *costs*-preference is not measured by the questionnaire but is uniformly set to '3'). Therefore the preference profile for strategy evaluation can be directly derived.

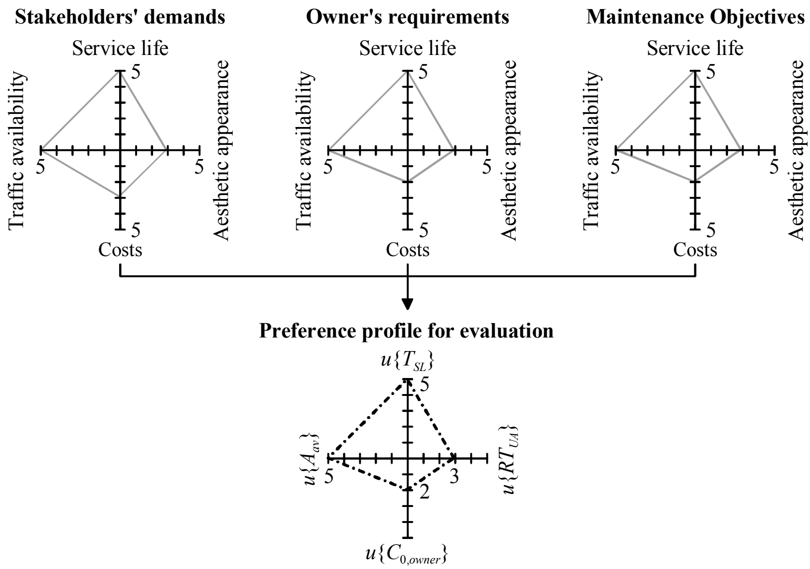



**Fig. 5.3:** Identification of the preference profile for the Köhlbrandbrücke.

In the case of the Brücke Bahrenfelder Chaussee (Fig. 5.4), the profiles of the demands, the requirements, and the objectives vary slightly: While there is not much demand for a long service life according to the stakeholders' demands, in the case of the owner's requirements and the maintenance objectives the bridge service life is rated with an intermediate value of '3'. It is also found that the economic performance is valued higher at the requirements level than on the objective level. As a compromise, the owner's requirements profile is chosen as the preference profile for strategy evaluation.

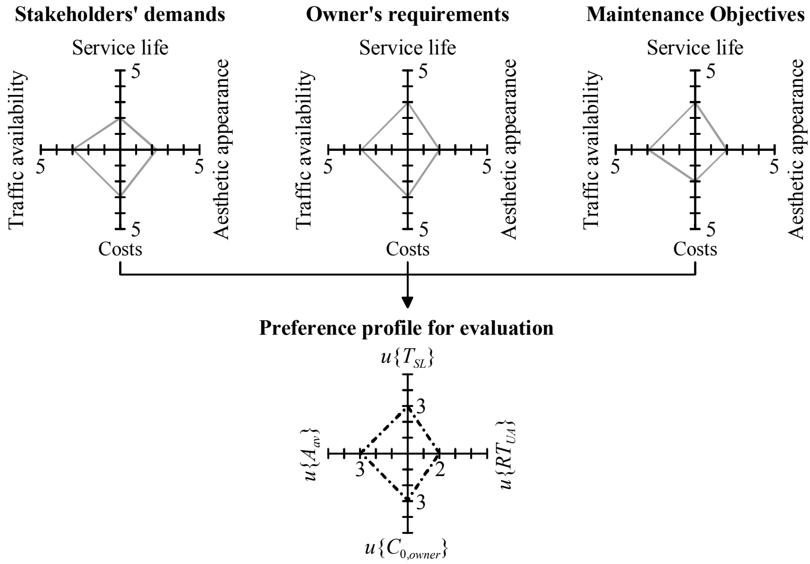



**Fig. 5.4:** Identification of the preference profile for the Brücke Bahrenfelder Chaussee.

## 5.4 Structural Representation

### 5.4.1 Overview

Ten individual SAMS-elements describe the load carrying structure as well as the bridge equipment and the concrete surface of the *Köhlbrandbrücke*, while in the case of the much smaller *Brücke Bahrenfelder Chaussee* four individual elements are sufficient for the investigation in the scope of the present thesis. As a first step, the load carrying structure is expressed in the form of representative cross sections, which describe regions that are highly significant in terms of structural safety—typically because of high load to resistance ratios or because of structurally significant deterioration exposure. These representative cross sections form the basis of the SAMS-elements. Then, as a second step, the bridge equipment and the concrete surface are represented by proportional allocation to the SAMS-elements.

### 5.4.2 Köhlbrandbrücke

#### Locations of the Representative Cross Sections

The choice of the representative cross sections follows from the findings of a detailed structural analysis that was part of an expertise conducted by the Institute of Concrete Structures of the Hamburg University of Technology (Sigrist and Krahwinkel 2008). As can be seen in the following Fig. 5.5, the structural analysis focused on regions of the bridge that, due to symmetry considerations, represent the western ramp (axes 118 to 113 and 104 to 100) and the major part of the eastern Ramp (axes 00 to 04). In addition, the t-intersection between the axes 32, 36, and 57 was subject to a detailed structural analysis within the expertise.

The critical cross sections for the definition of the SAMS-elements are chosen in such a way that those cross sections with the minimum degree of fulfilment with respect to shear or bending are selected. In addition, the axes 118, 100, and 00 are chosen as representative cross sections because of their potentially high deterioration at the expansion joints.

The representative SAMS-elements are defined in such a way that each element contains one critical cross section. For example, the cross section at the axis 101 is characterised by the lowest ratio $R_d/S_d$ with respect to both negative bending and shear/torsion, whereas the axis 101.5 represents the most critical positive bending action compared to the bending resistance. Besides, the cross section at the axis 118 represents the special situation at an expansion joint. The following Fig. 5.5 illustrates the structural representation of the Köhlbrandbrücke.

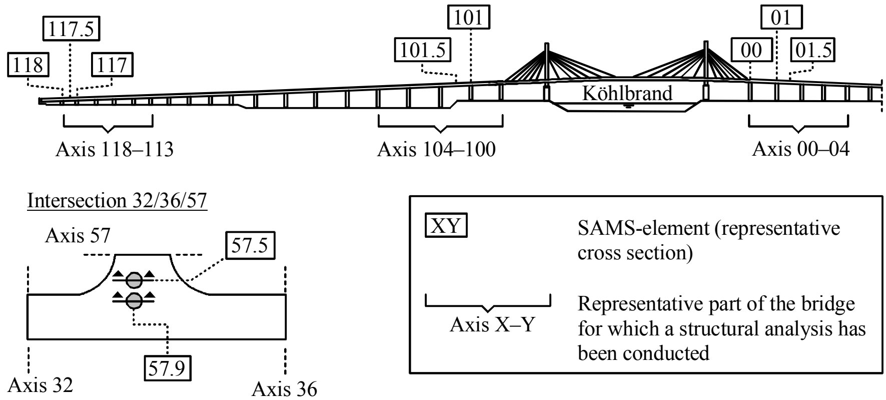



**Fig. 5.5:** Structural representation of the Köhlbrandbrücke by ten elements.

**Simplified Cross Sections and Internal Forces**

In contrast to the detailed structural analysis that was conducted during the expertise (Sigrist and Krahwinkel 2008), where the resistances to bending moments and shear were calculated with the help of detailed geometrical modelling of the cross sections, the present thesis applies structural models with simplifying geometries for the calculation of the cross sectional resistances (Fig. 5.6).

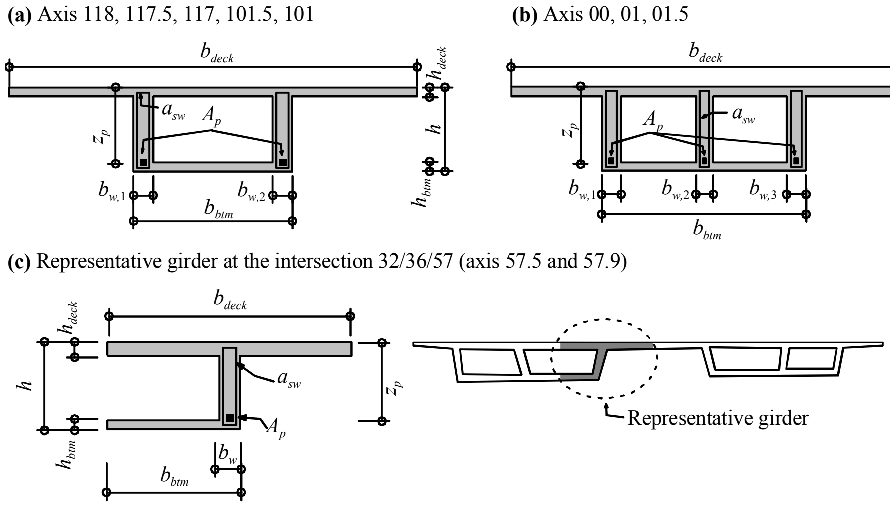



**Fig. 5.6:** Simplified critical cross sections, which are applied in the SAMS-model of the Köhlbrandbrücke: (a) Western ramp; (b) regular cross section at the eastern ramp; (c) representative girder of the multi-cell cross section at the intersection.

In general, three simplifying adaptations of the real cross sections are applied. First, the simplified cross sections are characterised by perpendicular webs as well as by horizontal decks and bottom slabs without any haunches. Second, only prestressed reinforcement (and optional FRP-strengthening) is considered for the calculation of the bending resistance, while a contribution of longitudinal mild steel reinforcement is not taken into account. Third, only evenly distributed shear reinforcement ($a_{sw}$ in mm²/m) is considered so that localised additional shear reinforcement is also expressed in this form.

These three simplifying adaptations are applied in such a way that the cross sections of the SAMS model show geometrical and mechanical properties, which are comparable to the real cross sections—especially in terms of the external dimensions (height and width) as well as in terms of the resulting bending and shear resistance. In the scope of the present thesis, this is achieved by setting the width of the deck and bottom slab as well as the overall height according to the design drawings. Then, the depth of the haunched slabs and webs are set to constant average values. The prestressing tendons are expressed in terms of a single resulting tendon and the shear reinforcement in the representative web is expressed in the form of a constantly distributed reinforcement with perpendicular stirrups. As a last step, the cross sectional areas of the prestressing steel $A_p$ and of the shear reinforcement $a_{sw}$ are adjusted in such a way that the resistances of the simplified cross sections are comparable to those found for the realistic models that have been investigated for the expertise on the Köhlbrandbrücke. In doing so, also the contribution of longitudinal mild steel reinforcement is indirectly accounted for by adapting the prestressing steel area.

The geometrical data, for example the amount of prestressing or mild steel reinforcement, as well as the data on material properties, such as regarding strength or elasticity, are taken from the expertise of Sigrist and Krahwinkel (2008) and from as-built drawings. Likewise, the internal forces, for example bending moment and shear force, are also taken from this expertise.

**Definition of SAMS-elements for Overall Structural Representation**

In the scope of the present case study the following SAMS-elements are defined for overall structural representation of the *Köhlbrandbrücke* according to sub-section 4.3.2 and 4.3.4; their geometrical dimensions as well as their weighting factors for the bridge equipment allocation can be found in Appendix III.

- The *span element 117.5*, which is characterised by the cross section in the span midway between axis 117 and 118, represents the axis range from 119 to 109 with its two spans of 50 m, seven spans of 42 m, and one shorter span of 34 m (Fig. 4.2 (a) ) in terms of the load bearing behaviour. In addition, this span element also represents the pavement and the concrete surface of the entire western ramp; both are allocated with a weighting factor $w_i$=0.32 according to the length of the western ramp which amounts to 1048 m compared to the overall length of 3218 m for the two ramps. It also represents the complete drainage system of the bridge, which is expressed in term $w_{Drain}$=1.00. A further differentiation for allocation, such as dividing the western pavement between the elements 117.5 and 101.5 is not applied mainly for the reason that the available data on deterioration progress as well as maintenance impact with respect to bridge equipment and concrete surface do not allow for differentiating between the more western and the more eastern part of the ramp.
- The *span element 101.5* represents the load bearing structure from axis 109 to 100 with respect to the cross section in the span midway between axis 101 and 102. Because bridge equipment as well as the concrete surface of the western ramp have been completely included by the above introduced element 117.5, no further allocations is made to this element.
- The *span element 01.5* represents the cross section that is located between the between axis 00 and 30 of the eastern ramp. Similarly to the above introduced element 117.5 of the western ramp, this SAMS element also receives the complete asphalt pavement and the concrete surface of the eastern ramp, which is expressed in terms of a weighting factor $w_i$=0.68 according to the length of the western ramp which amounts to 2170 m compared to the overall length of 3218 m for the two ramps.
- The *span element 57.5* describes the specific situation at the t-intersection between the axes 32, 36, and 57 with respect to positive bending (Fig. 5.5). However, no further allocation of equipment and concrete surface is made to this element.

- The *support element 118* describes the specific situation at the expansion joint in axis 118, which has been found to be a focus of high deterioration (e.g. WTM 1997). In addition to the load bearing structure at this location, it represents the expansion joint and the bearings at axis 118. In the case of the expansion joint, a weighting factor of $w_{Exp}$=0.125 is considered, which follows from the relation of one joint to the total number of eight expansion joints. The bearing at axis 118 is considered by a value of $w_{Bear}$=0.02, which reflects a rounded value based on the total number of seventy supports along the entire ramps.
- The *support element 117* stands for the supports in the axis range from 119 to 109 (without 118). It represents the hogging bending conditions over the pier at axis 117. Because only one expansion joint is located in this axis range, the respective weighting factor amounts to $w_{Exp}$=0.125. Analogously, the 10 supports in the axis range from 119 to 109 lead to a weighting factor of $w_{Bear}$=0.14.
- The *support element 101* is introduced in order to account for the high negative bending action over the piers in the axis range from 108 to 100, where the western ramp is found to span 70 m between the piers. The one expansion joint at axis 100, at the connection to the stay-cable bridge across the Köhlbrand, is considered by $w_{Exp}$=0.125; the 9 supports in the axis range from 108 to 100 lead to a rounded weighting factor of $w_{Bear}$=0.11.
- The *support element 00* represents the particular load bearing behaviour at the notched support of the 65 m span at this axis. Also the one expansion joint at axis 00 ($w_{Exp}$=0.125) and one support ($w_{Bear}$=0.03) are considered.
- The *support element 01* describes the high negative bending action over the piers in the axis range from 01 to 29, where the eastern ramp is found to span 55 to 50 m between the piers. While no expansion joint is located within this axis range, a number 29 supporting piers amounts to a weighting factor of $w_{Bear}$=0.41.
- Finally, the *support element 57.9* describes the specific situation at the t-intersection between the axes 32, 36, and 57 with respect to negative bending and shear (Fig. 5.5). It also represents the remaining bearings of the eastern ramp, which encompasses the supporting piers from axis 30 to 42, as well as the expansion joints at axis 15, 30, 57, and 42 in terms of weighting factors $w_{Exp}$=0.5 and $w_{Bear}$=0.29, respectively.

### 5.4.3 Brücke Bahrenfelder Chaussee

#### Locations of the Representative Cross Sections

The load carrying behaviour of the bridge *Brücke Bahrenfelder Chaussee* is represented by four cross sections. This representation is chosen on the basis of a simplified structural analysis, which indicated that sufficient representation with respect to positive bending is given by taking into account only the shorter western span. This is because in the western span a significantly lower degree of fulfilment occurs, compared to the longer eastern span. The shear is accounted for by two individual elements because of the different exposition characteristics at the expansion joint and at the intermediate support. The following Fig. 5.7 illustrates the structural representation.

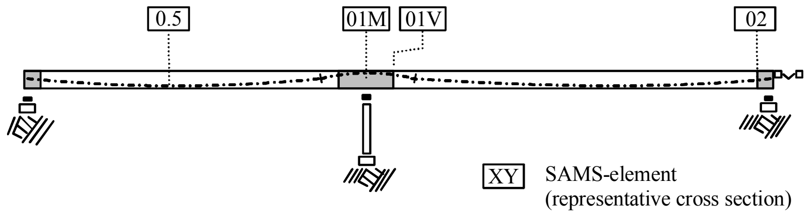



**Fig. 5.7:** Structural representation of the Brücke Bahrenfelder Chaussee.

### Simplified Cross Sections and Internal Forces

The simplified box-girder cross sections for SAMS-modelling are defined following the same simplification criteria as for the *Köhlbrandbrücke*. The data regarding geometrical and material properties—except for the ultimate stress of the FRP-strengthening material—are based on the documentation of the bridge *Bahrenfelder Chaussee* (LSBG 2010). The values for the internal forces have been calculated based on a simple beam model representation with the help of the structural design software *RStab* which takes into account the haunched cross sections with their varying geometric properties.

### Definition of SAMS-elements for Overall Structural Representation

As it is the case with the Köhlbrandbrücke, detailed information on the structural representation in terms of geometry, material properties, internal forces, as well as the weighting factors $w_i$ are listed in Appendix III. In the scope of the actual case study the general structural representation by SAMS-elements is achieved, according to sub-section 4.3.2 and 4.3.4, as follows.

- The *span element 0.5*, which is characterised by the cross section at a distance of 11.99 m from axis *00*, represents—in addition to the load bearing behaviour at this location—the entire pavement, the drainage system, and all bearings of the bridge.
- The *support element 01M* with its characteristic cross section at axis 01 represents the bending behaviour at the intermediate support.
- The *support element 01V* with its characteristic cross section at a distance *h* east from the cross-beam at axis 01 represents the shear behaviour at the intermediate support.
- The *abutment element 02* with its characteristic cross section at a distance *h* west from the cross-beam at axis 02 represents the shear behaviour at axis 02, which shows to be more critical than at axis 00. In addition, it also represents the condition development of the only expansion joint and of the entire concrete surface of the bridge.

## 5.5 Deterioration Mechanisms and Maintenance Activities

### 5.5.1 Overview

For both case study bridges, all deterioration processes that are described in chapter 4 are considered for strategy simulation. For many of these processes, quantitative parameter adaptation can be achieved on the basis of bridge inspection results, following the procedure that is described in section 4.6.

In the case of the *Köhlbrandbrücke*, altogether 56 different maintenance activities are defined in order to account for locally restricted maintenance activities, for example kerb/sidewalks rehabilitation at the western ramps only or expansion joint repair at selected locations. Such a local maintenance application practice, as it has indeed been observed for the Köhlbrandbrücke, accounts for the large bridge-dimensions. These dimensions, on the one hand lead to variations in the deterioration rates and maintenance needs, such as slower concrete surface deterioration at the western ramp compared to the eastern ramp. On the other hand, these dimensions cause long durations of the individual maintenance activities, such as pavement replacement works proceeding from the eastern ramp to the western ramp, so that local differentiation of the works within the SAMS program helps to reproduce the gradual performance improvement—first the eastern, then western ramp—within the simulation. Because of its less complex structure, the investigation of the Brücke *Bahrenfelder Chaussee* considers a smaller number of 27 different maintenance activities. The whole range of considered activities for both case studies is listed in Appendix III.

However, in contrast to the deterioration mechanisms, the data-basis for adjusting the maintenance parameters is less comprehensive so that bridge specific quantitative parameter adjustment could only be achieved for a limited number of activities. For example, concrete repair at the outside of the girders has been applied to both bridges; however, in the case of the Köhlbrandbrücke this was the case only around the time of the latest bridge inspection so that no data with respect of post-maintenance deterioration could be recovered. In contrast, concrete repair, together with protective coating was applied to the *Brücke Bahrenfelder Chaussee* in 1988, so that the post–intervention condition development could be analysed for adapting the maintenance effect parameters for these activities.

Considering the above described limitations, the parameter values with respect to the physical maintenance effects on deterioration progress are estimated on the basis of the above described δ-curves that illustrate the deterioration progress (see section 4.6). This is achieved by comparing the deterioration rates before and after maintenance application. When more than one maintenance activity is applied at the same time, the maintenance effects are allocated to the activities based on deliberate assumptions. Where this procedure is not possible due to lacking bridge inspection data or due to the fact that the activity has not been applied at all, the parameter values are being chosen on the basis of the relevant literature. Similarly, information from the maintenance log regarding the application costs is referred to where applicable. However, reliable estimates for the traffic flow effects cannot be derived from the available case study data. In such cases, where no direct reference data from the case study bridges can be found, the parameter values must be estimated on the basis of the relevant literature.

The main data source for adjusting the physical deterioration and maintenance parameter values is provided by the reports of the *regular bridge inspections*. These provide descriptions of the found defects (type and location) as well as recommendations for mid-term maintenance and rehabilitation needs. For the *Köhlbrandbrücke*, the inspection reports of the years 1984, 1987, 1991, 1998 and 2007 are used for adjusting the deterioration parameters of bridge equipment and the concrete surface. In cases where it was found that the regular bridge inspection activities spanned between one to two years, the above reference years are referring to the beginning year; in case the inspection covered more than two years, the reference year was defined as the medium year of the inspection activities. In the case of the *Brücke Bahrenfelder Chaussee* the inspection reports of the years 1976, 1982, 1985, 1988, 1994, 2000, 2006, and 2009 and are used as an information source.

Also, in the case of the *Köhlbrandbrücke*, reports on *special investigations*, which refer to localised damages to prestressing tendons and to results from concrete strength tests with drilled cores, are considered. They are used for adjusting the deterioration parameters of steel corrosion and concrete strength deterioration to the actual development.

The data for adjusting the economic parameter values have been collected during interviews with the bridge managers and also by review of the bridge files, which have been kindly be provided for reference by both bridge authorities. As it is described in sub-section 4.7.4 (equation 4.60), the cost value of each activity is transformed to the year of bridge completion in order provide for a common time basis for all cost input parameters. This is the year 1974 in the case of the *Köhlbrandbrücke* and 1970 for the *Brücke Bahrenfelder Chaussee*. The transformation is achieved with reference to the construction price index of the German *Statistisches Bundesamt* (2011: Table 5).

However, it must be emphasised that this data analysis method for adapting the deterioration and maintenance parameters relies on simplifying and idealising assumptions. Particularly in the case of the Köhlbrandbrücke, the interpretation of the inspection results for the bridge equipment is based on the general verbal bridge inspection reports and their rather qualitative defect descriptions. With respect to the enormous bridge dimensions, the condition rating—as it is applied in the scope of the present thesis—has a considerable intuitive element. The same is generally true for the Brücke Bahrenfelder Chaussee—however with comparatively less an intuitive element because of the smaller bridge size and because of the more systematic rating system according to the German RI-EBW-PRÜF (2007, 2013), which is applied by the managing authority for this bridge.

### 5.5.2 Parameter Adaptation: Deterioration and Maintenance effects

**Prestressing Steel: Corrosion and Maintenance Effects**

An expertise on the condition of the *Köhlbrandbrücke*, which has been conducted in 2008, provides a general assessment of the intensity of corrosive attack on prestressing tendons at different locations (Sigrist and Krahwinkel 2008: 51–52; Dette 2008a: 70). The defects that have been found during special inspections (Strom- und Hafenbau 1986b/c/d; Strom- und Hafenbau 1988a) indicate that the corrosion intensity is the highest at the poorly compacted tendon low points at the intersection of the web with the bottom slab. At these intersections, chloride containing water from leaking drainage pipes poses considerable danger and corrosive attack has been detected at several locations over the last years (Sigrist and Krahwinkel 2008: 19; Dette 2008a: 5, 9). In contrast to this, the corrosion intensity in the deck slab—when it was exceptionally found at transverse tendons (Wayss & Freytag 1994)—appears to be clearly lower in comparison to the tendon low-points. However, the expertise also points out that in spite of the only minor attack, which has so far been detected at these locations, the high chloride-contamination of the deck slab must be kept in mind. In particular, the results of potential field measurements, which have been conducted in 2007 by the ibac-institute of the RWTH Aachen University as a subinvestigation for the expertise, indicate possible corrosion activity at the connection between web and deck slab (ibac 2007, 2007a).

Generally, the values for the deterioration parameters $T_i$ and $\alpha_1$ are estimated based on the detected corrosive loss. Because the corrosive losses are defined for only one point in time—the time of damage detection—one of the two parameter values must be defined so that the other can be calculated. This is achieved in the scope of the present thesis by first estimating the initiation time $T_i$ and then calculating the deterioration rate $\alpha_1$ from the detected corrosive loss. In the scope of the present investigation, tendon corrosion is assumed to follow a simple bi-linear curve so that the further parameters $\alpha_2$ and $\alpha_3$ as well as $\delta_{k,1}$ and $\delta_{k,2}$ are not numerically defined.

*High intensity tendon corrosion* (H) is assumed to occur at the tendon low points of the eastern ramp of the *Köhlbrandbrücke*, where it has been detected at two locations. The first major tendon corrosion had been detected in the year 1986 and is located in the span between axis 50 and 52 (axis 51 does not exist) of the approach ramp *Breslauer Rampe* of the *Köhlbrandbrücke* (Strom- und Hafenbau 1986a). The second case of considerable tendon corrosion had been detected in the year 1984 in the span 4 between axis 4 and 5 at the interior web of the two-cell box girder (Strom- und Hafenbau 1987, 1993). Both defects are characterised by poor compaction (honeycombing) of the concrete around the tendons and corrosion due to accumulation of chloride contaminated water within the box girder, which has possibly entered through leaking cable entries of the road lightning. Therefore the initiation time $T_i$ is estimated to be only two years on average because of the very poor protection in combination with chloride-attack. This estimate for the initiation time is based on the corrosion model that is described in the above sub-section 4.4.2. There, the expected initiation time is two years for a concrete cover of 50 mm, which is representative for the outer tendons of the *Köhlbrand-brücke* with their damaged concrete and high chloride exposition. On this basis, the deterioration rate $\alpha_1$ can be calculated from the corrosive loss that has been found (Table 5-1). In the scope of the present investigation, an average deterioration rate of $\alpha$=0.006 is assumed to apply to the tendon low points of the entire eastern ramp, which is the mean value of the two rates in Table 5-1. This rate is plausible in the case of high intensity prestressing steel corrosion (see Table 4-7). The upper and lower $\alpha_1$-values of the high-intensity value distribution are defined in analogy to the example from sub-section 4.4.2 (Table 4-5). The upper and lower values for the initiation time are chosen to 0 and 4 years, as opposed to 1 and 6 years in Table 4-7, in order to define a symmetrical interval with a mean value of 2 years. Both parameter values are listed in the below Table 5-2.

**Table 5-1:** Estimation of corrosion rates

| **Span** | **Detection** | **Corrosive loss** | **Initial amount** [a] | **δ** | **ΔT** [b] | $\alpha_1$**=(1-δ)/Δ*T*** |
|---|---|---|---|---|---|---|
| [—] | [year] | [Strands] | [Strands] | [—] | [a] | [‰] |
| 4/5 | 1984 [c] | 8.5 [d] | 264 | 0.968 | 8 | 4.0 |
| 50/52 | 1986 | 65.5 [e] | 858 | 0.924 | 10 | 7.6 |

[a] Total number of prestressing strands in the web where the defect has been found.

[b] Time elapsed since construction (1974) minus an estimated initiation period $T_i$ of two years

[c] Exposed tendons at this location have been reported as early as in the bridge inspection report of 1984 (Strom- und Hafenbau 1984). However, the explicit information on the number of corroded and broken strands is taken from a later memo (Strom- und Hafenbau 1993).

[d] Six broken strands and five strands with corrosive loss (the latter are accounted for by 50%)

[e] 28 broken strands and 75 strands with corrosive loss (the latter are accounted for by 50%)

*Medium intensity tendon corrosion* (M) is assumed to occur at the tendon low points of the western ramp. Here, the deterioration intensity is assumed to be lower compared to the eastern ramp because noteworthy corrosive attacks have not been observed so far. Also the longitudinal tendons at the high-points and at the bridge supports are assumed to deteriorate with medium intensity. Due to the fact that field data from bridge inspection are not available for these locations, the deterioration parameters are defined according to Table 4-5 and Table 4-7 from sub-section 4.4.2 with small adjustments to the values in order to provide non-overlapping and symmetrical value ranges for the intensity intervals. The initiation time $T_i$ is assumed to range from 4 years to 50 years with a modal value of 27 years, which is assumed on the basis of the above described maximum initiation time for high intensity corrosion and the maximum expected initiation time for medium intensity prestressing steel corrosion and a concrete cover of 50 mm in Table 4-5. The modal value of 27 years, however, is assumed on the basis of a concrete cover of 80 mm. This is assumed because in the scope of the present investigation, medium intensity corrosion refers both to the low points and the high points, which show a higher concrete cover. The corrosion rate $\alpha_1$ is chosen—with minor adjustments—according to Table 4-7 to range from 0.5 ‰ to 2.5 ‰ (2.6 ‰ in Table 4-7).

*Low intensity tendon corrosion* (L) is not considered within the SAMS-model of the *Köhlbrandbrücke*. However, in order to define a complete set, the values for the deterioration parameters $T_i$ and $\alpha_1$ are taken from the example in sub-section 4.4.2 (Table 4-5 and Table 4-7) with small adjustments.

The following Fig. 4.12 illustrates the deterioration progress, as it is found for the field data from the eastern ramp as well as the intensity level boundaries that follow from the parameter value distributions that are listed in Table 5-2.

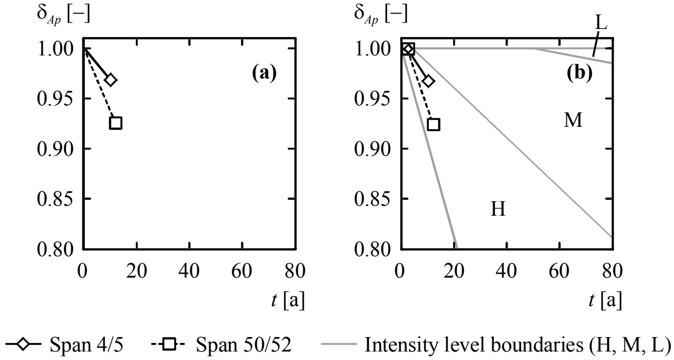



**Fig. 5.8:** Prestressing steel corrosion and associated deterioration intensity levels (H, M, L) for the *Köhlbrandbrücke*: (a) Field data with respect to the corrosive loss that has been found in span 4/5 and span 50/52 according to the above Table 5-1; (b) Adjusted corrosion progress (with initiation time of 2 years) compared to the chosen intensity levels (H, M, L).

In the case of the *Brücke Bahrenfelder Chaussee* no detailed data for tendon corrosion could be found at all. Therefore, the same deterioration parameter values are used for this bridge as for the *Köhlbrandbrücke*.

**Table 5-2:** Initiation times and deterioration rates of prestressing steel for three different intensity levels: high (H), medium (M), and low (L).

| | **H** | | | **M** | | | **L** | | |
|---|---|---|---|---|---|---|---|---|---|
| | max | mod | min | max | mod | min | max | mod | min |
| $T_i$ | 0 | 2 | 4 | 4 | 27 | 50 | 50 | 200 | 1000 |
| $\alpha_1$ [‰] | 9.5 | 6 | 2.5 | 2.5 | 1.5 | 0.5 | 0.5 | 0.25 | 0 |

*Note*: In the scope of the present investigation, tendon corrosion is assumed to follow a simple bi-linear curve so that $\alpha_1 = \alpha_2 = \alpha_3$ and $\delta_{k,1} = \delta_{k,2} = 0$.

The following maintenance activities are considered to delay the tendon corrosion progress:

- *Intensified routine maintenance* prevents the ingress and accumulation of chloride containing water due to regular cleaning of the drainage system and regular minor patch repair of concrete and asphalt pavement.
- *Hydrophobic impregnation of girders* reduces the ingress of water and chlorides into the concrete.
- *Concrete coating of girders* reduces the ingress of water and chlorides into the concrete around the tendons located in the girders. However, this activity does not significantly reduce tendon corrosion at the high-points over the piers because at this location the corrosive attack comes through a leaking asphalt pavement.
- *Concrete repair of girders* is effective by reducing the ingress of water and chlorides into the concrete around the tendons. This is done by restoring surface defects, such as spalling and cracks, and by removing chloride contaminated concrete and correcting inbuilt deficiencies, such as honeycombing in concrete. This activity is not effective at high-points over the piers where corrosive attack mainly comes through a leaking asphalt pavement
- *Pavement resurfacing and replacement* also is effective by reducing the ingress of water and chlorides into the concrete around the tendons. However, the effect is assumed to be restricted to the tendon high points over the piers.
- *Bridge deck reconstruction* is effective analogously to pavement resurfacing.
- *Drainage overhaul* also is effective by reducing the ingress of water and chlorides into the concrete around the tendons. However, the effect is assumed to be restricted to the tendons at the low points where water from leaking pipes can accumulate and cause corrosion at tendons that are surrounded by poorly compacted concrete.
- *Expansion joint repair or replacement* is only effective for the tendons that are actually adjacent to the joints. The maintenance effect results from reducing the ingress of water and chlorides through the expansion joints.
- *Tendon conservation* is highly effective by applying anti-corrosive treatment and sealing with protective repair mortar.

However, the quantitative effects of these maintenance activities on the tendon corrosion process could not be derived from the available inspection and maintenance data and therefore must be estimated in the scope of the present investigation. This is because none of the maintenance activities that reduce tendon corrosion, such as concrete coating or tendon conservation, have been applied between successive defect measurements. Therefore no deterioration reduction factors $\nu$ could have been derived from the inspection data for these mechanisms. Also no detailed cost data for the tendon conservation activities have been. Likewise, in the case of the Brücke Bahrenfelder Chaussee no quantitative maintenance effects on tendon corrosion could be derived from the inspection data and the maintenance logs. As a consequence, the values for the deterioration parameters (ν, δ, Δδ), the traffic flow reduction ($\nu_{traf}$, $dt_{traf}$), and the activity costs ($C_{app}$) are estimated in the scope of the present thesis. A detailed listing of the estimated parameter values can be found in Appendix III.

**Mild Steel Shear Reinforcement: Corrosion and Maintenance Effects**

In the scope of the present thesis, only the corrosion of the shear reinforcement in the webs is taken into account, while the structural effects of mild steel corrosion on the deck or the bottom slab are not considered.

In the case of the *Köhlbrandbrücke*, the most severe cases of corroding shear reinforcement, which became apparent by spalling of the concrete cover, are located at the box girder inside. These were caused by leaking drainage components or leaking cable entries, which led to chloride attack (Fig. 5.9). Other observed cases of shear reinforcement corrosion on the girder outside may also be caused by chloride ingress in the spray zone of the carriageway. Carbonation on the other hand is unlikely to be the cause, as the field measurements in 2007 and 1994 showed surprisingly low carbonation depths in the range of 0–2 mm in combination with a 5%-percentile value for the concrete cover of the web mild steel reinforcement in the range of 1.5 cm (span 115–116) to 3 cm (span 00–01) (ibac 2007, 2007a; WTM 1997).

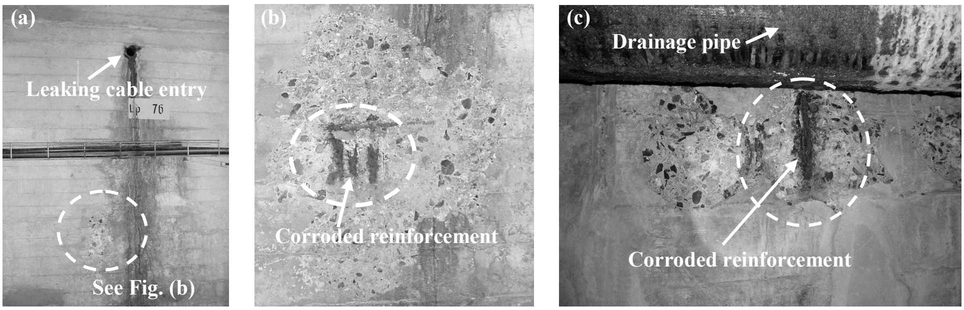



**Fig. 5.9:** Spalling due to reinforcement corrosion: (a) leaking cable entry with visible spalling / delaminated concrete (see mark); (b) close-up of the before mentioned damage after removing the lose material; (c) spalling due to reinforcement corrosion next to a leaking drainage pipe. (Photos: HPA 2007)

Unfortunately, no explicit data have been reported on the extent of corrosion, such as percentage loss in cross section or mass. Therefore, the values for the deterioration parameters $T_i$ and $\alpha_1$ are estimated according to Table 4–5 (c=25mm) and Table 4-7 with some adjustments to the values in order to provide symmetrical value ranges for the intensity intervals; here the most significant adjustment is a modal value for $T_i$ of 11 years in contrast to 4 years in Table 4-5 simply for reasons of symmetry.

However, in the case of the *Köhlbrandbrücke*, a general plausibility of the estimated parameter values follows from a visual estimation of the extent of corrosion at the two example damages of Fig. 5.9. These two damages have been newly detected during the major inspection 2006/2007. The corrosive loss after 32 years (1974–2006) is visually estimated to be roughly 20% for the single bar. Thus, under the assumption that every second bar is corroded by such an extent, the δ-value is 0.9. It can be seen in the below Fig. 5.10 that the observed deterioration can be described as a medium-intensity corrosion process.

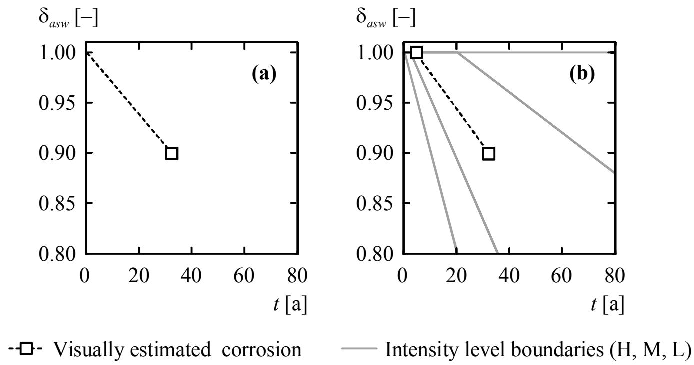



**Fig. 5.10:** Shear reinforcement corrosion and associated deterioration intensity levels (H, M, L) for the *Köhlbrandbrücke*: (a) Corrosion progress that follows from visual estimation (Fig. 5.9); (b) Adjusted corrosion progress (with initiation time of 4 years) compared to the theoretically derived intensity levels (H, M, L).

Also for the *Brücke Bahrenfelder Chaussee* no detailed data for mild steel corrosion could be found. Therefore, for this bridge the same deterioration parameter values are used as for the *Köhlbrandbrücke*. As with tendon corrosion, mild steel corrosion is assumed to follow a simple bi-linear curve so that the further parameters $\alpha_2$ and $\alpha_3$ as well as $\delta_{k,1}$ and $\delta_{k,2}$ are not defined.

**Table 5-3:** Initiation times and rates of the shear reinforcement corrosion for three different intensity levels: high (H), medium (M), and low (L).

| | **H** | | | **M** | | | **L** | | |
|---|---|---|---|---|---|---|---|---|---|
| | max | mod | min | max | mod | min | max | mod | min |
| $T_i$ | 0 | 1 | 2 | 2 | 11 | 20 | 20 | 30 | 1000 |
| $\alpha_1$ [‰] | 10 | 8 | 6 | 6 | 4 | 2 | 2 | 1 | 0 |

*Note*: In the scope of the present investigation, shear reinforcement corrosion is assumed to follow a simple bi-linear curve so that $\alpha_1 = \alpha_2 = \alpha_3$ and well $\delta_{k,1} = \delta_{k,2} = 0$.

The following maintenance activities are considered as having an influence on shear reinforcement corrosion progress by reducing the ingress of water and chlorides into the concrete:

- intensified routine maintenance
- hydrophobic impregnation of girders
- concrete coating of girders

- concrete repair of girders
- drainage overhaul
- expansion joint repair / replacement
- shear reinforcement conservation

However, as it is the case with tendon corrosion, no maintenance parameters ($\nu$, $\delta$, $\Delta\delta$, $\nu_{\text{traf}}$, $dt_{\text{traf}}$, $C_{\text{app}}$), could have been derived from the inspection and maintenance data for these mechanisms. Therefore, the parameter values are estimated in the scope of the present thesis (see Appendix III).

**FRP-Strengthening Systems: Deterioration and Maintenance Effects**

To both case study bridges, FRP-strengthening has not been applied so far. Thus, the deterioration parameter values are chosen from the relevant literature. This is done in such a way that the deterioration curve which is applied in the SAMS-program corresponds with the deterioration curve of the tensile strength of the FRP material as it is found by Wilcox for the FRP system type B (Wilcox: 2008: 41). A more detailed description of FRP-deterioration can be found in sub-section 4.4.3.

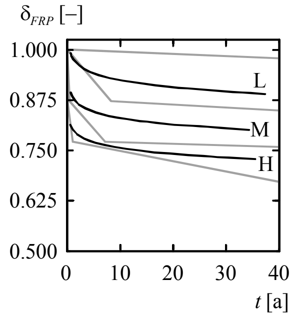



**Fig. 5.11:** Deterioration of FRP strengthening systems according to Wilcox (2008: 41, FRP system type B) visualised by black curves and associated deterioration intensity levels H, M, L with grey lines marking the intensity boundaries.

**Table 5-4:** Initiation times and rates of the FRP deterioration

| | **H** | | | **M** | | | **L** | | |
|---|---|---|---|---|---|---|---|---|---|
| | max | mod | min | max | mod | min | max | mod | min |
| $T_i$ | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| $\alpha_1$ | 0.55 | 0.375 | 0.15 | 0.15 | 0.1 | 0.01 | 0.01 | 0.0075 | 0.001 |
| $\alpha_2$,$\alpha_3$ | $\alpha_2/\alpha_1$=0.05 and $\alpha_3/\alpha_2$=0.025 for all intensity levels | | | | | | | | |
| $\delta_{k1}$, $\delta_{k2}$ | $\delta_{k1}$=0.875 and $\delta_{k2}$=0.775 for all intensity levels | | | | | | | | |

In contrast to the above described corrosion mechanisms, FRB deterioration is assumed not to be affected by any of the considered maintenance activities.

**Concrete Strength: Development and Maintenance Effects**

As it is described in sub-section 4.4.4, there are two opposite developments of the concrete strength. The first one regularly appears in the form of an increase due to continuing hydration, while the second one can potentially occur as a deterioration-induced decrease. The second one of these effects—strength deterioration—is considered for the two case study bridges because both are affected by alkali-aggregate reaction (Sigrist 2004, VdZ 1973, Strom- und Hafenbau 1973). In this case the increase in concrete strength is included in the model representation because the reduction factors already implicitly account for this effect (see sub-section 4.4.4).

The prestressed concrete ramps of the *Köhlbrandbrücke* are vulnerable to alkali-aggregate reaction (AAR) along their whole length. In fact, typical damages that are related to AAR have constantly developed since completion. The AAR-susceptibility with an alkali level of up to 3.6 kg/m³ had been detected already during the construction phase (VdZ 1973). But the application of low-alkali cement, which would be technically indicated at a level above 3.0 kg/m³ (e.g. Bödeker 2003: 35–36), was not considered to be necessary because the concrete bridge structure was thought to be sufficiently protected from water by the carriageway pavement and also by the cantilevering deck, which was expected to shield the webs from rainfall (Strom- und Hafenbau 1973). However, the hoped-for structural protection did not suffice: soon after construction—as early as in the year 1975—typical AAR-related damages occurred in the form of efflorescence, exudations of alkali-silica gel, and pop-outs (Strom- und Hafenbau 1975 and 1976). During the following years, more and more damages occurred, especially in the form of AAR-induced map cracking at the cross beams at expansion joints (Strom- und Hafenbau 1984, 1987a; WTM 1997). These damages developed continually over the years and even today, more than 30 years after completion, the concrete is still susceptible to AAR (Strom- und Hafenbau 1988; TUHH 2005).

The following Fig. 5.12 shows the results from in-situ strength test, which have been conducted at various locations. The detailed data from the in-situ strength tests, which applied diverse methods such as cores with different diameters and rebound hammer tests, are taken from Schwab (1975), Strom- und Hafenbau (1986d, 1988a, 1993), Wayss & Freytag (1994), WTM (1997) and TUHH (2005); the data analysis under consideration of the different test methods and core diameters has been conducted by Dette (2014).

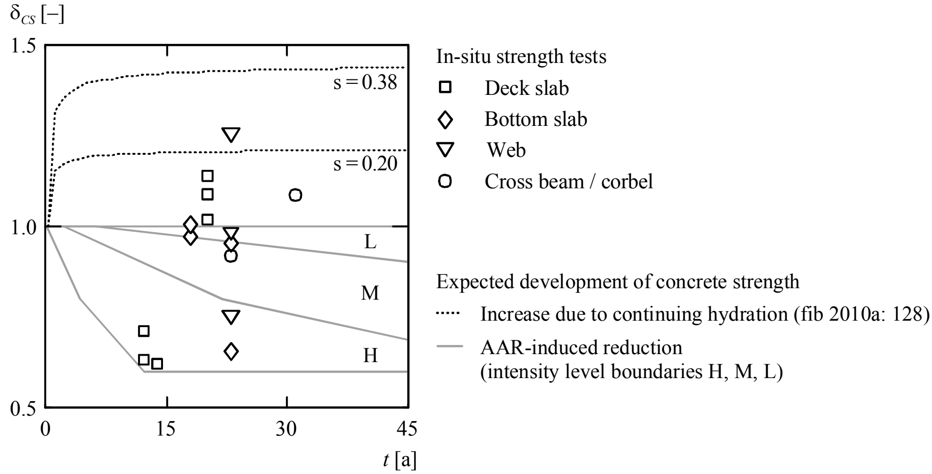



**Fig. 5.12:** In-situ strength tests in comparison to the expected development of concrete strength in terms of an increase as well as in terms of AAR-induced reduction (Dette 2014).

The results of the in-situ strength tests show that the development of concrete strength clearly lags behind the expected increase as it occurs for sound concrete (see sub-section 4.4.4 and equation 4.33 and 34): With only one exception, all tests reveal lower results than are expected for the post-hardening characteristics of the cement type PZ 450 F, which has been used within the concrete mixture (Strom- und Hafenbau 1973) and hence should result in a strength increase along the curve for the coefficient s=0.2. Moreover, even significant reductions in compressive strength can be found.

A plausible explanation for these reductions can be found in the form alkali-aggregate reaction (see sub-section 4.4.4). However, considerable differences in the test results, which presumably result from local particularities and thus individual exposition conditions, complicate the identification of specific deterioration courses and of characteristic locations were they refer to. First, surprisingly the data show a very high reduction in the compressive strength of the deck at the Breslauer Rampe (axis 50-54) over the first 12 to 14 years after completion, while the tests of drilled cores from the deck of the same ramp show an increase of strength above the initial value only six years later. A possible explanation for this is that the cores represent different exposition conditions. Secondly, a considerable scattering of the data occurs 23 years after construction, with considerable differences in the development of the concrete strength. In the case of the web this may be explained by the fact that the comparatively high value of 61.5 MPa—meaning an increase by 26% compared to a mean compressive strength of 49 MPa—is found at a location of 45 cm above the bottom slab. This is a location which is exposed to only minor chloride contamination. However, the scatter in the 1997-values of the bottom slab cannot be explained in this way. An example for the strength development at locations with low chloride contamination is found for the specimens taken from the bottom slab in axis 4.5 (Strom- und Hafenbau 1993); here virtually no strength reduction is found. However, at the same time the expected strength increase for sound concrete is not observed.

In spite of this somewhat mixed picture, AAR-induced strength reduction is considered in the scope of the investigation of the Köhlbrandbrücke. This is done by defining three deterioration intensity classes on the basis of the strength test results for taking into account different exposition to chloride and water ingress as well as the resulting deterioration rates and initiation times. In detail, the bridge deck concrete is assumed to be affected by high intensity deterioration, as the results from 1986 and 1988 imply, while the bottom slab is generally assumed to be affected by low-intensity AAR. The webs are assumed to be exposed to medium or high intensity in the vicinity of expansion joints while AAR is assumed to be of medium concern at other locations. As can be seen by comparison with Fig. 4.14 (c), the estimated deterioration development is—in its general shape—consistent with the approximate δ-t-relationships that are developed on the basis of relevant literature with the only significant difference being the slower deterioration rates. However, it must be kept in mind that the assumed deterioration intensity classes and the resulting deterioration parameter values at different structural components (e.g. web and bottom slab) must be regarded as a quite rough representation which certainly does not account for all particularities.

**Table 5-5:** Alkali-aggregate-reaction: Values for the deterioration parameters for three different intensity levels: high (H), medium (M), and low (L).

| | **H** | | | **M** | | | **L** | | |
|---|---|---|---|---|---|---|---|---|---|
| | max | mod | min | max | mod | min | max | mod | min |
| $T_i$ | 0 | 1 | 2 | 2 | 4 | 6 | 6 | 8 | 10 |
| $\alpha_1$ | 0.05 | 0.03 | 0.01 | 0.01 | 0.00625 | 0.0025 | 0.0025 | 0.00125 | 0.0 |
| $\alpha_2, \alpha_3$ | $\alpha_2/\alpha_1$=0.30 for all intensity levels | | | | | | | | |
| $\delta_{k1}$, $\delta_{k2}$ | $\delta_{k1}$=0.80 and $\delta_{k2}$=0.70 for all intensity levels | | | | | | | | |

For the *Brücke Bahrenfelder Chaussee*, the detailed investigation by Sigrist (2004) showed that this bridge is susceptible to alkali-aggregate reaction. However, the same deterioration parameter values as for the *Köhlbrandbrücke* are assumed within the SAMS-model because no detailed data for concrete strength deterioration could be found for the Bahrenfelder bridge.

The following maintenance activities are considered as having an influence on AAR by reducing the ingress of water and chlorides into the concrete or by replacing AAR-susceptible concrete, as in the case of bridge deck reconstruction.

- intensified routine maintenance
- hydrophobic impregnation of girders

- concrete coating
- concrete repair of girders
- pavement resurfacing and replacement (concrete of the deck slab only)
- bridge deck reconstruction (concrete of the deck slab only)
- drainage overhaul (concrete of the webs and bottom slab only)
- expansion joint repair / replacement

The available data from inspection reports or the like, however, do not allow for adapting the parameter values (ν, δ, Δδ) of maintenance activities such as concrete repair, hydrophobic impregnation, or coating to the specific context of the case study bridges. This is because on the one hand, in the case of the *Köhlbrandbrücke*, the available data for test cores refer to different locations within the bridge, and thus do not allow for analysing maintenance effects on deterioration progress. On the other hand, for the *Brücke Bahrenfelder Chaussee,* data on strength development are even not available at all. Consequently the values for the maintenance effect parameters are estimated in the scope of the present thesis (see Appendix III).

**Concrete Surface Condition: Deterioration and Maintenance Effects**

In contrast to corrosion and concrete strength development, the available data from inspection reports and other data sources allow for a detailed analysis of concrete surface deterioration development over time for both case study bridges. Thus, parameter adaptation can be achieved, both for the deterioration mechanisms and the maintenance effects.

In the case of the *Köhlbrandbrücke*, the inspection reports of the years 1991–1992, 1987, and 1984 as well as 1999 and 2006 are evaluated in terms of $CI_{CS}$-values with respect to the overall description of the extent of concrete defects. From these qualitative descriptions a generally steady development of the damage development can be concluded. Additionally, the major rehabilitation of the kerbs and sidewalks of both the eastern and western ramp during the year 1989 are accounted for by resetting the $CI_{CS}$-value for the kerbs and sidewalks to 1.0. Based on this linear transformation, the concrete surface deterioration for the bottom slab, the webs, the deck cantilever, and the sidewalks at the four different elements that represent the concrete surface (117.5, 101.5, 1.5, and 57.5) is depicted in the following Fig. 5.13 (a)/(d).

Similarly, the bridge inspection reports of the *Brücke Bahrenfelder Chaussee*—referring to the years 1971, 1976, 1982, 1985, 1988, 1994, 1998, 1999, 2000, 2001, 2006, and 2009—are analysed with respect to the extent of cracking, breaking, and leakage related deficiencies. Also the major bridge overhaul in 1988, which included concrete repair works and coating, is considered by resetting the $CI_{CS}$-value to 1.0 for all concrete surfaces of the entire bridge. The resulting concrete deterioration is depicted in Fig. 5.13 (g). Compared to the Köhlbrandbrücke, the available data also allowed for separately considering the concrete deterioration of the piers and abutments.

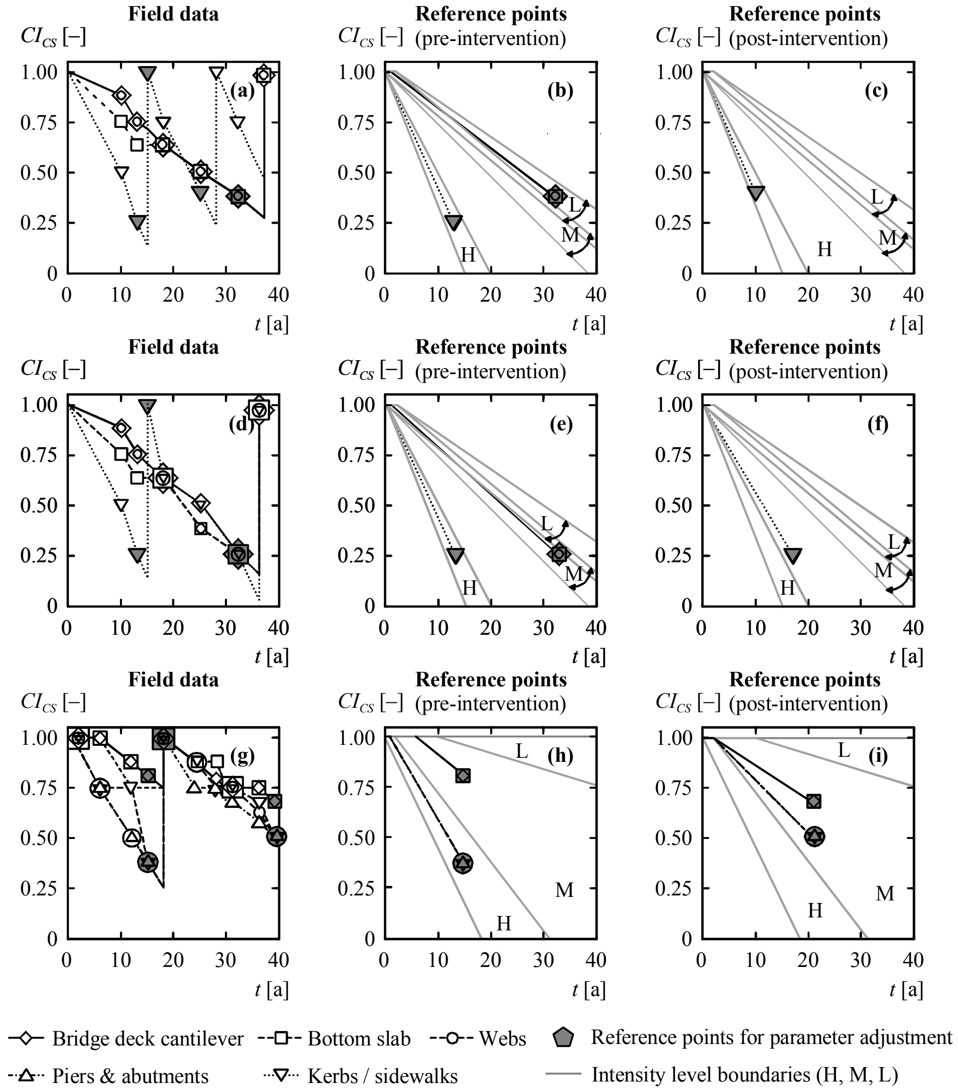



**Fig. 5.13:** Concrete surface condition development and associated deterioration intensity levels (H, M, L). (a) – (c) Western ramp of the *Köhlbrandbrücke*: (a) Condition development derived from inspection and maintenance data; (b) Development from the time of bridge construction; (c) Development after maintenance intervention; (d) – (f) Eastern ramp of the *Köhlbrandbrücke* and (g) – (i) *Bahrenfelder Chaussee* with an analogous representation of the condition development (Data source: Dette 2014).

As it occurs in Fig. 5.13, the deterioration development shows characteristic patterns for the different structural components (e.g. webs, bottom slab, deck cantilevers, kerbs, piers): For both bridges, the kerbs and sidewalks show the highest deterioration rates. Also the webs in the case of the Brücke Bahrenfelder Chaussee show very high deterioration rates, while in the case of the Köhlbrandbrücke the web-deterioration is comparably slower and is only in the medium intensity range. The piers and abutments are only investigated for the Brücke Bahrenfelder Chaussee and are also characterised by considerably high deterioration progress over time. Medium deterioration intensity occurs for both bridges in case of bottom slab. Slow deterioration progress occurs for both bridges in terms of deck cantilever deterioration.

With reference to the grey shaded reference points in Fig. 5.13, the deterioration parameters are adjusted to the inspection data. The resulting parameter values are listed in Table 5-6 (Köhlbrandbrücke) and Table 5-7 (Brücke Bahrenfelder Chaussee). The intensity-intervals in the case of the Köhlbrandbrücke are defined as very narrow intervals in order to differentiate between the deterioration of the eastern and western ramp. At the same time the intervals are defined in such a way that the observed condition development is taken as reference for defining the sample mean values for the SAMS parameter distributions. By doing so, the medium- and low-intensity intervals partially overlap each other (see Fig. 5.13 (b), (c), (e), (f)).

**Table 5-6:** Deterioration data of the concrete surface (Köhlbrandbrücke) for three different intensity levels: high (H), medium (M), and low (L).

| | **H** | | | **M** | | | **L** | | |
|---|---|---|---|---|---|---|---|---|---|
| | max | mod | min | max | mod | min | max | mod | min |
| $T_i$ | 0 | 0 | 0 | 0 | 1 | 2 | 0 | 1 | 2 |
| $\alpha_1$ [‰] | 66 | 58 | 50 | 26 | 24 | 22 | 22 | 20 | 18 |

**Table 5-7:** Deterioration data of the concrete surface (Bahrenfelder Chaussee) for three different intensity levels: high (H), medium (M), and low (L).

| | **H** | | | **M** | | | **L** | | |
|---|---|---|---|---|---|---|---|---|---|
| | max | mod | min | max | mod | min | max | mod | min |
| $T_i$ | 0 | 1 | 2 | 2 | 6 | 10 | 10 | 15 | 20 |
| $\alpha_1$ [‰] | 54 | 44 | 34 | 34 | 21 | 8 | 8 | 4 | 0 |

The following maintenance activities are considered as having an influence on concrete surface deterioration:

- intensified routine maintenance
- hydrophobic impregnation
- concrete coating
- concrete repair and sidewalk rehabilitation

For both bridges, the available deterioration data allow for estimating the effect of concrete repair and coating activities on the deterioration progress. In the case of the Brücke *Bahrenfelder Chaussee* the reduction factors ν can be derived for all structural components: girder, sidewalks/kerbs, webs, bottom slab, and deck cantilever. However, because coating has been applied together with concrete repair, the effects of both activities cannot be allocated independently. For simplification, it is assumed that concrete repair only leads to a restoration of the concrete surface, while coating also results in a reduction of the deterioration rate. This reduction factor ν is found to be 1.0 for a period of two years, followed bei a reduction of ν =0.4 for the webs, the kerbs, and the piers/abutments and a reduction of ν=0.18 for the bridge deck cantilever and the bottom slab, both being in effect over a period of at least 19 years (Dette 2014). This reduction in deterioration rate is easily understood by comparing the slopes of the deterioration curves in Fig. 5.13 (e) an (f).

For the *Köhlbrandbrücke* only rehabilitation of the kerbs/sidewalks could be investigated; here, the findings suggest that this activity leads—in addition to a reconstitution of the original as-new condition—to a deterioration rate reduction by 24% in the case of the eastern ramp, while no deterioration rate reduction was found in the case of the western ramp (Dette 2014).

The other two activities—intensified routine maintenance and hydrophobic impregnation—have not been applied for the case study bridges. Therefore parameter adjustment is not possible and the maintenance effect parameter values are estimated in the scope of the present thesis (see Appendix III).

**Asphalt Pavement Condition: Deterioration and Maintenance Effects**

For the *Köhlbrandbrücke*, the development of the pavement condition is established from the defect listings of the regular bridge inspections between 1984 and 2007. Additionally, the major bridge deck reconstruction works of the years 1987–1990, which included a replacement of the asphalt pavement, is taken into account by resetting the asphalt condition to a $CI_A$-value of 1.0 after reconstruction.

Similarly, the asphalt condition development in the case of the *Brücke Bahrenfelder Chaussee* is established on the basis of the reports from the inspection that took place between 1970 and 2009. Also, the pavement resurfacing works in 1988 and 2006 are considered by setting the asphalt back to as-new condition ($CI_A$ =1.0).

For both bridges, this is done by rating the four defect types (surface defects, surface deformations, potholes, and cracking) and deriving the resulting asphalt condition. The resulting developments are depicted in the following Fig. 5.14 (a) and the estimated values for the deterioration parameters are listed in Table 5-8 and Table 5-9.

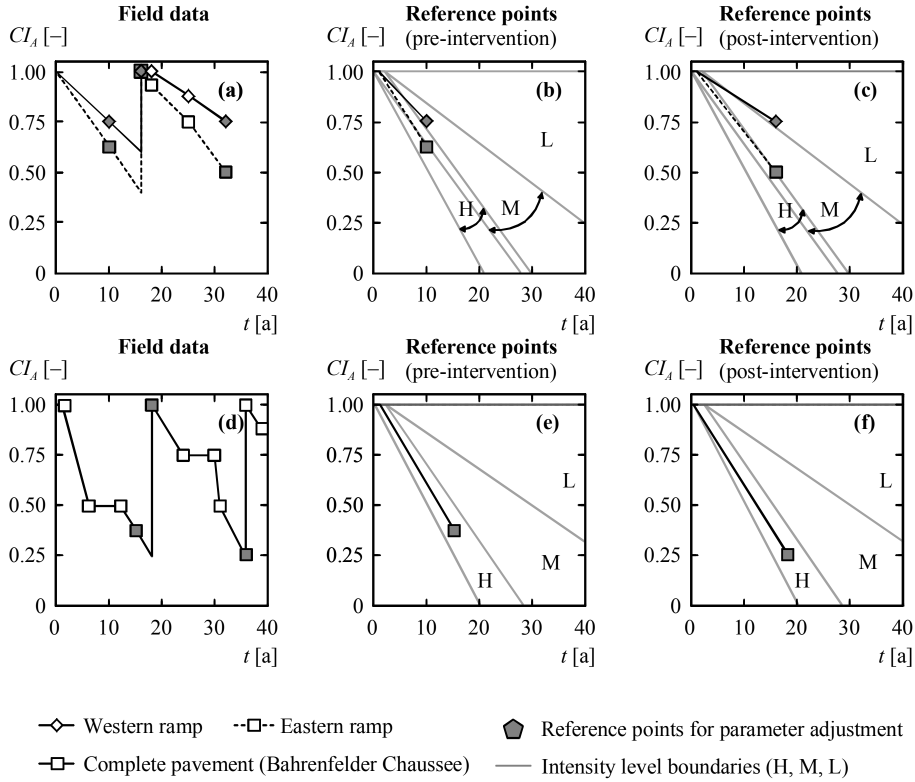



**Fig. 5.14:** Asphalt pavement condition development and associated deterioration intensity levels (H, M, L). (a) – (c) *Köhlbrandbrücke*: (a) Condition development derived from inspection and maintenance data; (b) Development from the time of bridge construction; (c) Development after maintenance intervention; (d) – (f) *Bridge Bahrenfelder Chaussee*: Analogous representation of the condition development (Data source: Dette 2014).

As it can be seen in Fig. 5.14 (b) and (c), the intervals that describe the high (H) and medium (M) deterioration intensities at the eastern and western ramp of the *Köhlbrandbrücke* do partially overlap. This is because—as explained above for the concrete surface deterioration—the intensity intervals are defined in the scope of the present investigation in such a way that the observed condition development is taken as reference for defining the sample mean values for the SAMS parameter distributions. In the observed case, the deterioration curves of the two ramps lie relatively close together and thus the intervals must overlap in order to provide for not too narrow intensity intervals

For both bridges, it occurs that the pavement service life, in terms of the time span between major interventions (about 15 years), lies in the range that is reported in the relevant literature (sub-section 4.4.6).

**Table 5-8:** Deterioration data of the asphalt pavement (Köhlbrandbrücke)

| | **H** | | | **M** | | | **L** | | |
|---|---|---|---|---|---|---|---|---|---|
| | max | mod | min | max | mod | min | max | mod | min |
| $T_i$ | 0 | 1 | 2 | 0 | 1 | 2 | 2 | 4 | 6 |
| $\alpha_1$ [‰] | 48 | 42 | 36 | 36 | 28 | 20 | 20 | 10 | 0 |

**Table 5-9:** Deterioration data of the asphalt pavement (Bahrenfelder Chaussee)

| | **H** | | | **M** | | | **L** | | |
|---|---|---|---|---|---|---|---|---|---|
| | max | mod | min | max | mod | min | max | mod | min |
| $T_i$ | 0 | 1 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| $\alpha_1$ [‰] | 50 | 44 | 38 | 38 | 28 | 18 | 18 | 9 | 0 |

In the scope of the present thesis, the following maintenance activities are considered to be effective in terms of pavement deterioration:

- intensified routine maintenance
- pavement resurfacing and replacement
- bridge deck reconstruction

Both resurfacing and pavement replacement are considered to lead to a condition improvement to $\delta$=1.0. In the case of the *Köhlbrandbrücke*, the pavement replacement, which was applied in combination with the bridge deck reconstruction works of the years 1987–1990, led to a 40% decrease in the deterioration rate along the pavement of the western ramp while the deterioration rate along the eastern ramp was only reduced by 20% after maintenance intervention. These effects were observed over a time span of 15 years, ranging from maintenance intervention until the bridge inspection in 2006 (Dette 2014). Also, an initiation period of one year (as a mean value) is assumed, according to the initiation period for the initial asphalt pavement (Table 5-8). The same effects—with respect to the asphalt pavement—are also assumed to apply for bridge deck reconstruction works, which also include a complete pavement replacement.

In the case of the *Brücke Bahrenfelder Chaussee*, a condition improvement to as-new condition could be observed after the pavement resurfacing works of 1988; the deterioration rate was reduced by 6% over a period of 18 years as a result of this intervention (Dette 2014). In contrast to the pavement replacement works, no initiation period is assumed because pavement resurfacing does not lead to a new pavement as replacement works do.

The activity intensified routine maintenance has not been applied for the case study bridges. Therefore parameter adjustment is not possible and the maintenance effect parameter values are estimated in the scope of the present thesis (see Appendix III).

**Bearing Condition: Deterioration and Maintenance Effects**

For both bridges, the data provided by the bridge inspection reports and other sources also allow for reliable parameter adjustment for deterioration and maintenance.

For the *Köhlbrandbrücke*, the as-observed development of the bearing condition is derived from the defect listings of the bridge inspection reports of the years 1991/1992, 1998/2000, and 2005/2007. Additionally, the replacement of the roller bearings from axis 30 to 42 by elastomeric bearings in 1997 as well as the repair of the broken roller bearings at axis 100 and 00 in the year 1984 are considered by resetting the δ-value to 1.0 for the replaced bearings. The resulting developments are depicted in the following Fig. 5.15. It can be seen that the roller bearings at axis 00 and 100 show considerably higher deterioration intensities than the bearings at other locations.

In the case of the *Brücke Bahrenfelder Chaussee*, the bearing condition development shows a remarkable long initiation period $T_i$ of 28 years, during which no defects have been found. This is followed by a rapid deterioration progress over 8 years, after which significant corrosion of the bearing has been found.

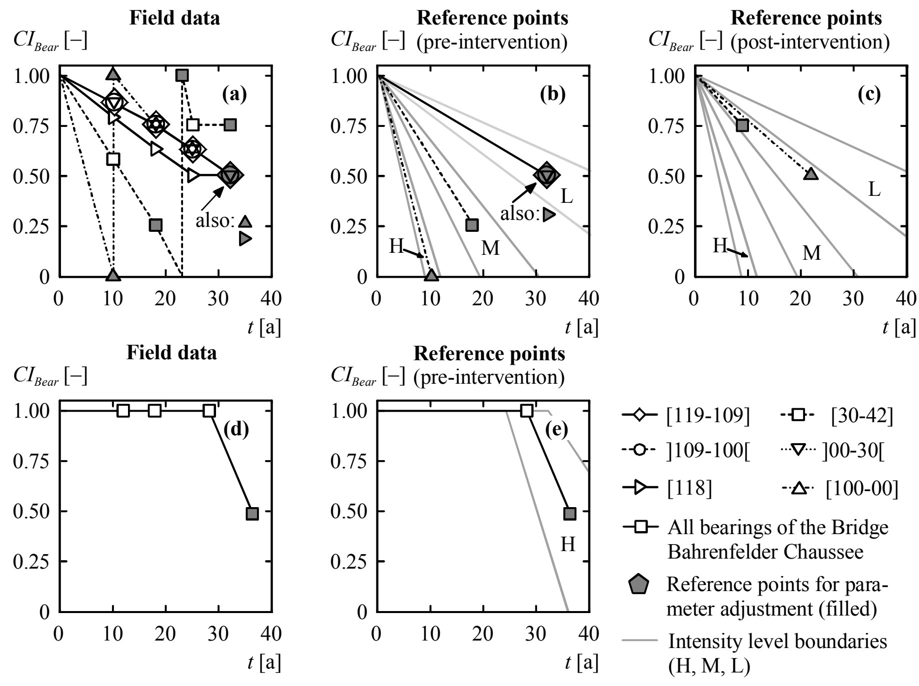



**Fig. 5.15:** Bearing condition development and associated deterioration intensity levels (H, M, L). (a) – (c) *Köhlbrandbrücke*: (a) Condition development derived from inspection and maintenance data; (b) Development from the time of bridge construction; (c) Development after maintenance intervention; (d) – (e) *Bahrenfelder Chaussee* with an analogous representation (Data source: Dette 2014)

For both bridges, the deterioration parameters $T_i$ and $\alpha_1$, as they are derived from the observed condition developments, appear plausible in the context of the data, which are reported in the literature: The $\alpha_1$-values for medium and low intensity bearing deterioration at the Köhlbrandbrücke fit into the value-ranges, which can be derived by linear approximation from the $T_{SL}$-data in sub-section 4.4.6. Only the quickly replaced roller bearings show a much higher deterioration rate than it should have been expected from experiences with other bridges. Also the values for $T_i$ and $\alpha_1$ in the case of the Brücke Bahrenfelder Chaussee seem plausible because they can be easily replicated by a bi-linear approximation of an exponential deterioration curve as in Fig. 4.17 (b).

**Table 5-10:** Deterioration data of the bearings (Köhlbrandbrücke)

| | **H** | | | **M** | | | **L** | | |
|---|---|---|---|---|---|---|---|---|---|
| | max | mod | min | max | mod | min | max | mod | min |
| $T_i$ | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| $\alpha_1$ [‰] | 115 | 100 | 85 | 53 | 43 | 33 | 20 | 16 | 12 |

**Table 5-11:** Deterioration data of the bearings (Bahrenfelder Chaussee)

| | **H** | | | **M** | | | **L** | | |
|---|---|---|---|---|---|---|---|---|---|
| | max | mod | min | max | mod | min | max | mod | min |
| $T_i$ | 24 | 28 | 32 | – | – | – | – | – | – |
| $\alpha_1$ [‰] | 85 | 62.5 | 40 | – | – | – | – | – | – |

*Note*: One intensity class (H) is sufficient because the bearings are represented by one single SAMS-element.

In the scope of the present thesis, only bearing repair and replacement are considered as having a significant maintenance effect on the bearing condition development.

For the *Köhlbrandbrücke*, it is found that the bearing repair in 1984 led to a considerable reduction in the deterioration rate in the range of 77% while the replacement of the roller bearings in axis 30–42 by elastomeric bearings led to a 33% reduction in the deterioration rate (Dette 2014).

In the case of the *Brücke Bahrenfelder Chaussee*, until today no bearing maintenance has been applied. Therefore, the maintenance effects of bearing repair and replacement is considered in terms of setting back the condition to the $\delta$-value to 1.0, while no effects on the deterioration rate are considered.

**Expansion Joint Condition: Deterioration and Maintenance Effects**

For both bridges, the development of the expansion joint condition can be reconstructed on the basis of the inspection reports as well as on the basis of the maintenance history. In particular, expansion joint repair and replacement has been applied for both bridges. Thus, also the maintenance effects on deterioration progress can be numerically estimated.

As for the *Köhlbrandbrücke*, all expansion joints have been repaired in the year 1988 together with the major deck reconstruction and asphalt replacement works. Further maintenance works took place in the years 1998–1999, where all joints, with the only exception of the joint at axis 119, have been subject to repair. For simplification, though, it is assumed in the scope of the present thesis, that also the joint at axis 119 has been repaired. Later on, during the year 2002, the joint in axis 100 has been subject to repair activities and during the year 2004 all joints except for axis 100 and 00 have been replaced. Again, for simplification it is assumed that all expansion joints along the entire bridge have been repaired in the year 2004. The resulting condition development of the expansion joints at different locations is depicted in the following Fig. 5.16 (a). For SAMS-parameter adaptation, the two maintenance alternatives repair and replacement are interpreted as a complete condition restoration of the affected joint.

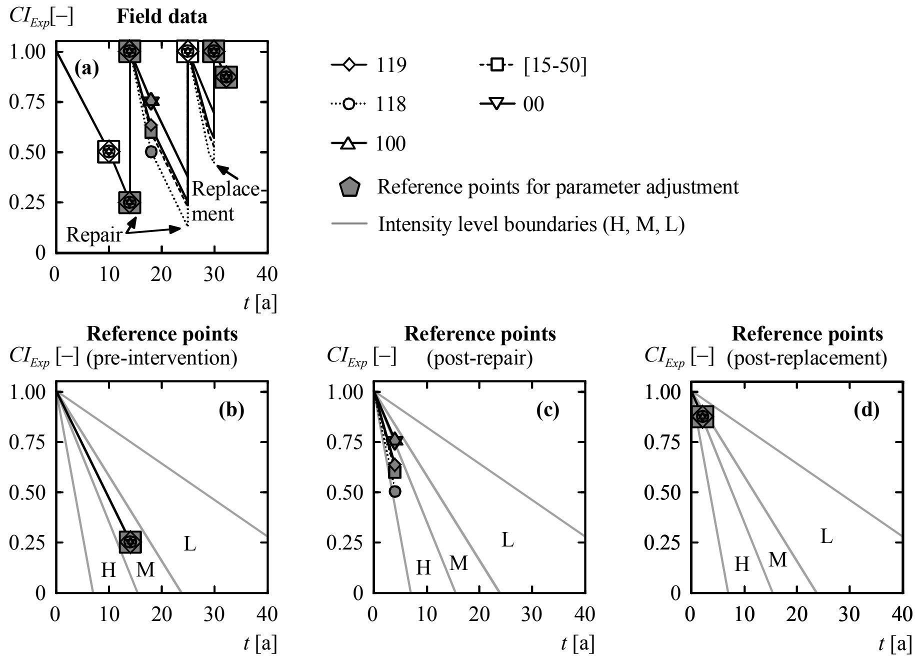



**Fig. 5.16:** Expansion joint condition development and associated deterioration intensity levels (H, M, L) in the case of the *Köhlbrandbrücke*: (a) Condition development derived from inspection and maintenance data; (b) Development from the time of bridge construction; (c) Development after expansion joint repair; (d) Development after expansion joint replacement. (Data source: Dette 2014)

In the case of the *Brücke Bahrenfelder Chaussee* two maintenance interventions took place: a complete replacement in 1988 and expansion joint repair in 1995. The resulting condition development under consideration of the interventions and the bridge inspections reports of the years 1982, 1985, 1988, 1995, 1998, 2000, 2001, and 2009 is depicted in Fig. 5.17.

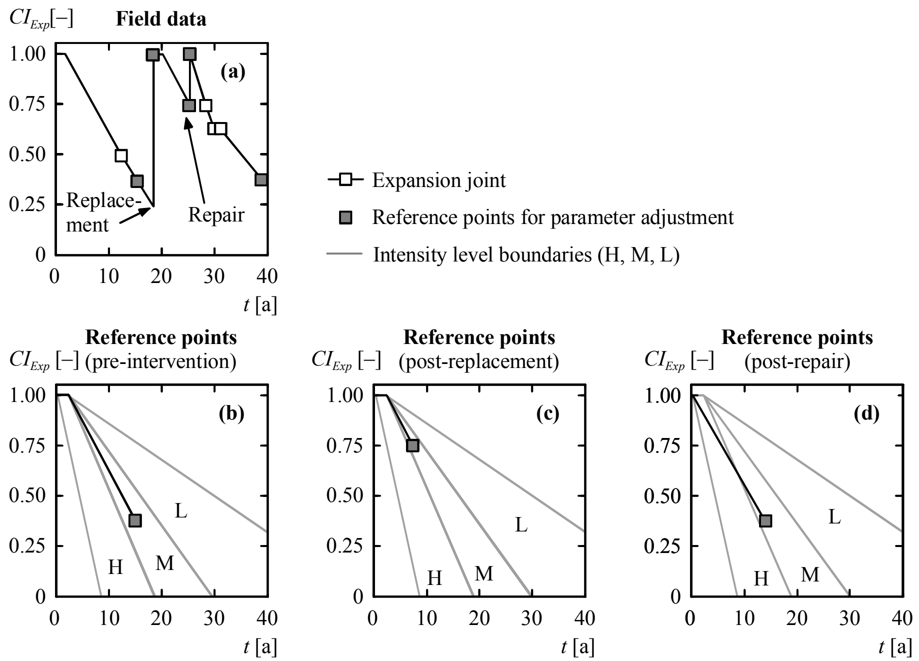



**Fig. 5.17:** Expansion joint condition development and associated deterioration intensity levels (H, M, L) for the *Brücke Bahrenfelder Chaussee*: (a) Condition development derived from inspection and maintenance data; (b) Development from the time of construction; (c) Development after first maintenance intervention; (d) Development after second intervention. (Data source: Dette 2014)

The deterioration parameters, as they follow from adaptation to the observed condition development, are listed in the following Table 5-12. These values fit into the $\alpha_1$-value ranges, which can be derived from the $T_{SL}$-data that are reported by other authors (sub-section 4.4.6).

**Table 5-12:** Deterioration data of the expansion joints

| | **Köhlbrandbrücke (M)** | | | **Bahrenfelder Chaussee (M)** | | |
|---|---|---|---|---|---|---|
| | max | mod | min | max | mod | min |
| $T_i$ | 0 | 0 | 0 | 2 | 2 | 2 |
| $\alpha_1$ [‰] | 65 | 53.6 | 42.2 | 60 | 48 | 36 |

*Note*: Here, only the values for the intensity class M are defined because all original joints of the *Köhlbrandbrücke* are found to deteriorate with a similar rate. In the case of the *Brücke Bahrenfelder Chaussee* only one joint exists so that rate differentiation is not necessary.

In addition to *intensified routine maintenance*, two more maintenance activities are assumed to affect the expansion joint condition deterioration:

- expansion joint repair
- expansion joint replacement

Both repair and replacement are considered to lead to a condition improvement to $\delta$=1.0, while the effects on the deterioration rate differ from bridge to bridge.

In the case of the *Köhlbrandbrücke*, the deterioration rate reductions $\nu$ for expansion joint repair, as they are found by comparing the pre-repair deterioration rates of Fig. 5.16 (b) with the post-repair rates of Fig. 5.16 (c), appear to be negative. This in effect means an increased rate (from 17% to 133%) after repair works. Even if this seems not completely implausible—repaired joints are exhibiting pre-existing damage and therefore may initially deteriorate faster than new joints—the deterioration rate should be expected to return to the basic rate after some time. Therefore, it is arbitrarily assumed that the deterioration rate increase lasts over a time span of four years. Similarly, the condition rating system implies a 17% increase in the deterioration rate after expansion joint replacement (Fig. 5.16 (d)). This seeming increase, however, is neglected in the scope of the present investigation. One reason for this is the short reference time span of two years (2004 to 2006), which may lead to an overestimated deterioration rate because small initial defects can cause condition devaluation over this short time interval. Furthermore, such an increase seems not plausible: Even if it may be that today joints decay faster, it is rather expected, that in the longer run the replaced expansion joints will show a similar deterioration rate like the replaced ones.

In the case of the *Brücke Bahrenfelder Chaussee*, it can be seen that the condition development after the expansion joint replacement, as in Fig. 5.17 (c), can be assumed to follow the same path as for the original joint, as in Fig. 5.17 (b). Expansion joint repair, on the other hand, is found to result in a deterioration rate reduction by 7%, which is assumed to be effective over 14 years according to the observed time interval from 1995 to 2009.

**Drainage System: Deterioration and Maintenance Effects**

For both bridges, the condition development of the drainage system can also be reconstructed on the basis of the inspection reports. Maintenance effect data, however, cannot be derived for both bridges because no drainage overhaul has been conducted during the observed time horizon.

In the case of the *Köhlbrandbrücke*, the drainage system is mentioned only in the inspection reports of the years 1992, 1999 and 2006. With respect to these data, a rather straight deterioration curve is found for the drainage system (Fig. 5.18 (a)).

In the case of the *Brücke Bahrenfelder Chaussee*, in contrast, several consecutive inspection reports describe exactly the same condition, so that the condition development resembles the form of step function. For parameter adaptation, however, also a linear condition decay is assumed (Fig. 5.18 (b)). The deterioration and parameter values for the two bridges are listed in Table 5-13.

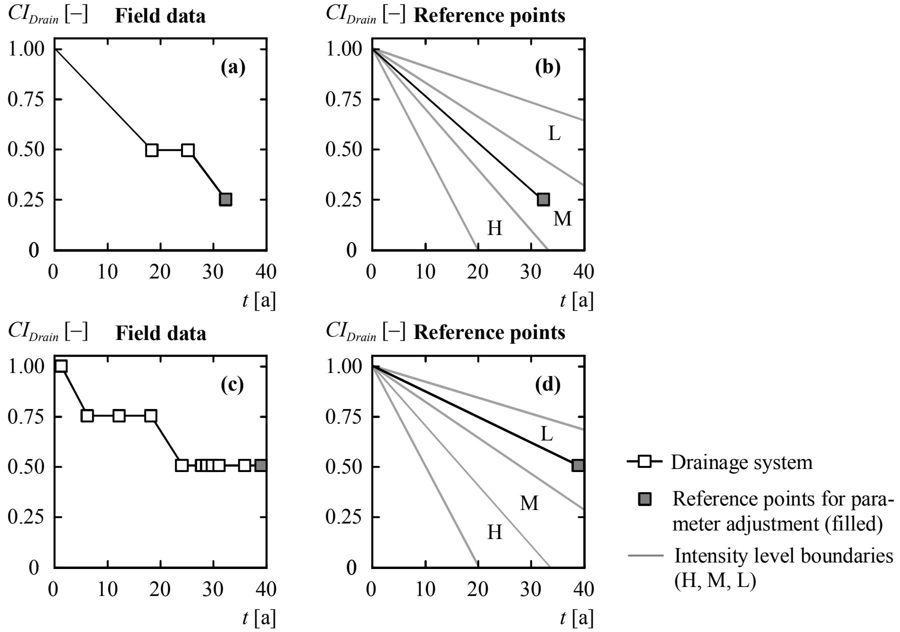



**Fig. 5.18:** Drainage system condition development and associated deterioration intensity levels (H, M, L): (a) – (b) *Köhlbrandbrücke*: (a) Condition development derived from inspection and maintenance data; (b) Simplified deterioration development that is used in the scope of the present investigation; (c) – (d) *Bridge Bahrenfelder Chaussee*: analogous representation. (Data source: Dette 2014)

The derived deterioration parameters are listed in the following Table 5-13. In the case of the *Köhlbrandbrücke*, the $\alpha_1$-value ranges fit to the ranges that can be derived from the $T_{SL}$-data that are reported by other authors (see sub-section 4.4.6). The deterioration rate in the case of the Brücke Bahrenfelder Chaussee appears to be very low, however it still seems plausible and not too far off from the expected range.

**Table 5-13:** Deterioration data of the drainage system

| | Köhlbrandbrücke | | | Bahrenfelder Chaussee | | |
|---|---|---|---|---|---|---|
| | max | mod | min | max | mod | min |
| $T_i$ | 0 | 0 | 0 | 0 | 0 | 0 |
| $\alpha_1$ [‰] | 30 | 23.5 | 17 | 18 | 13 | 8 |

*Note*: Here, only the values for the intensity class for each bridge are listed because for both bridges, the drainage system deteriorates with one single rate along the whole bridge length.

In addition to *intensified routine maintenance*, only the maintenance activity *drainage system overhaul* is assumed to affect the deterioration progress in the scope of the present thesis. However, no field data are available for deriving bridge specific parameter values. This is because no such interventions have taken place during the observed time horizon—neither in the case of the *Köhlbrandbrücke*, nor in the case of the *Brücke Bahrenfelder Chaussee*.

### 5.5.3 Maintenance Activities: Adaptation of Physical and Economic Parameters

**Intensified routine minor maintenance**

This activity is assumed to further reduce—in excess of regular minor maintenance—the deterioration rate for a number of structural components. In the scope of the present investigation, this effect is expressed in terms of $\nu_{1/2}$ values ranging from around 10% (high intensity effect) to around 0.5% (low intensity effect). High intensity maintenance effects are assumed to apply to the pavement and the expansion joints, which both benefit the most from routine minor maintenance, as it is explained in sub-section 4.5.2. Medium intensity effects are expected for the bridge deck in terms of reduced AAR-related concrete strength deterioration, and also for the concrete surface deterioration of the webs, as well as for the deterioration of the bearings and the drainage system. Only minor effects are assumed to apply for steel corrosion and concrete strength reduction as well as surface deterioration of the webs. In the SAMS algorithm, the intensified routine minor maintenance is defined in terms of an annually repeated activity.

Even if the routine maintenance measures are preferably conducted during off-peak hours, an average reduction of the traffic flow by 0.2% in addition to normal intensity maintenance is assumed to be the case ($\nu_{traf}$=0.002). This additional reduction is equivalent to three hours of traffic flow reduction by 25% every 14 days (3/24×0.25×1/14). The average additional annual costs are estimated to 0.28 € per m² of bridge deck surface, which follows from a basic cost value for routine maintenance provided by Vollrath and Tathoff (2002: 44) and an estimated 50% increase in maintenance costs due to the *higher intensity*. In the scope of the present thesis, the cost data that are provided by Vollrath and Tatthoff are assumed to refer to 2002 prices and are transferred to the time of bridge construction according to the construction price index of the German *Statistisches Bundesamt* (2011, Tab. 5).

The detailed maintenance parameter values that are used in the scope of the present investigation—referring to the physical effects, the traffic impact as well as the economic impact—are listed in Appendix III.

**Hydrophobic Impregnation of Concrete**

Because impregnation has not been applied for the two case study bridges, the $\nu_{1/2}$ and $dt_{1/2}$ values are estimated by considering the data estimates by Denton (2004: 47) and Pommerening et al. (2008: 36–38).

Here, the high and medium intensity value ranges—with mean values for the maintenance effect parameters of $\nu_1/\nu_2$=0.90/0.4 and effective periods of around two and six years respectively—are assumed to apply to treated concrete surfaces of the kerbs/sidewalks, the webs, and the bottom slab. The concrete surface deterioration of the cantilever soffits, however, is assumed to be reduced only in the medium range because the main cause for concrete surface deterioration at this location is found to be water and chloride attacking from above through a leaking pavement and conduits (Sigrist & Krahwinkel 2008: 39–42, Dette 2008a: A4-57– A4-60). Also medium range effects, which differ from high intensity effects in terms of a lower $\nu_2$-value, apply to AAR-induced concrete strength reduction in the webs and the bottom slab as well as to mild steel reinforcement corrosion. Low intensity effects are assumed to apply to prestressing tendon corrosion, expressed in terms of mean values for $\nu_1$ and $\nu_2$ in the range of 0.625 and 0.5 respectively, being effective over the same time periods as the other intensity classes (see Appendix III). It is assumed that hydrophobic impregnation cannot be applied at element-condition values below $CI_{CS}$=0.375, which describe a fair to poor condition with some breaking defects and cracking (Appendix II), in order to provide for sufficient substrate conditions for impregnation. In the case of insufficient surface conditions, concrete repair works must be applied before hydrophobic impregnation.

In the case of bridges where the concrete surfaces are not accessible from the ground level, the application of the surface treatment requires truck-mounted platforms or scaffolding. In this case the average reduction of the traffic flow is estimated to 25% for the duration of the maintenance work. The average costs are estimated to a rounded value of 10 €/m² based on Pommerening et al. (2008: 36). As Pommerening et al. do not specify a reference year for these costs, it is assumed in the scope of the present thesis that the costs refer to the year 2008. Additional costs for truck-mounted platforms or scaffolding are taken into account, which are freely estimated to be around 100 k€ for the *Köhlbrandbrücke* and 25 k€ for the *Brücke Bahrenfelder Chaussee* (see Appendix III).

**Coating of the Concrete Surface**

In the scope of the present thesis, coating is assumed to result in a temporary halt to deterioration progress, which is followed by a period with a reduced deterioration rate. However, the available data for the two bridges allow a parameter adaption only for the effects on the concrete surface deterioration of the *Brücke Bahrenfelder Chaussee* (see 5.5.2). For the concrete surface of the *Köhlbrandbrücke*, as well as AAR-induced strength reduction and steel corrosion at both bridges, no parameter adaption can be achieved. This is because concrete coating has been exclusively applied to the *Brücke Bahrenfelder Chaussee*, where in contrast no data are available with respect to AAR or steel corrosion.

Also, detailed information on the effect of surface coating has not been found in the relevant literature. The only information found in this context is provided by Pommerening et al. (2008: 36–38), who assign a twice as long service life (12 years) to a coating itself compared to hydrophobic impregnation (6 years)—however, this does not provide data on the actual maintenance effect. Therefore, the parameter values for the coating effects—except for the case of concrete surface deterioration of the Brücke Bahrenfelder Chausseee—are assumed in the scope of the present thesis with reference to the data for hydrophobic impregnation under the assumption that coating is more effective.

In order to provide for sufficient substrate conditions, it is assumed that coating cannot be applied at element-condition values below $CI_{CS}$=0.375, which describes a fair to poor condition with some breaking defects and cracking (Appendix II).

As it is the case for impregnation, the application of the coating may require truck-mounted platforms. In this case the average reduction of the traffic flow is estimated to 25% for the duration of the maintenance work. The average costs are estimated to 18 €/m² at 2008-prices according to Pommerening et al. (2008: 36) with additional costs for truck-mounted platforms or scaffolding.

The detailed maintenance parameter values that are used in the scope of the present investigation—referring to the physical effects, the traffic impact as well as the economic impact—are listed in Appendix III.

**Concrete Repair**

With respect to the concrete surface condition, concrete repair is assumed to result in a complete restoration to the original condition, whereas no reduction of the deterioration rate is assumed. In contrast to this, concrete repair is assumed to result in a temporary slowdown of shear reinforcement and tendon corrosion as well as of AAR-induced concrete strength reduction. In this case, the numerical effects in terms of $\nu_{1/2}$ and $dt_{1/2}$ values are estimated on the basis of Denton (2004: 48) who considers the effect of concrete repair on the corrosion induced reduction of structural safety in terms of a triangularly distributed temporary stop of deterioration progress. The effect in terms of a reduced corrosion rate and a slowing down of the deterioration of the concrete strength is estimated for the two intensity levels M ($\nu_{1/2}$ = 0.95/0.25 and $dt_{1/2}$ = 2/5) and L ($\nu_{1/2}$ = 0.625/0.25 and $dt_{1/2}$ = 3/6), while the effect on the concrete surface condition is defined in terms of high-intensity (H) effect (see Appendix III).

The concrete repair works may require sophisticated underbridge platforms. In this case the average reduction of the traffic flow is estimated to 25% for the duration of the maintenance work. The costs are estimated on the basis of field data that have been provided by the bridge owning authorities of the two bridges. The detailed data for both traffic flow impact and maintenance costs are listed in Appendix III.

**Kerb and Sidewalk Rehabilitation**

Analogously to the above described concrete repair works, this activity returns the kerbs and sidewalks back to as-new condition ($\delta_n$=1), while it does not reduce the condition deterioration rate—except for the eastern ramp of the *Köhlbrandbrücke*, where a reduction by 24%

could be observed (see 5.5.2). Further effects on other structural elements and deterioration mechanisms are only considered in terms of a reduced concrete strength reduction of the deck slab (see Appendix III).

The kerb and sidewalk repair can require sophisticated platforms and or scaffolding so that the average reduction of the traffic flow is estimated to 25% for the duration of the maintenance work. In the case of the *Köhlbrandbrücke*, the costs can be estimated on the basis of field data in the listings of the actually applied activities and their costs, while in the case of the Brücke Bahrenfelder Chaussee a cost estimate on the basis of Pommerening et al. (2008) is applied (Appendix III).

**Pavement Resurfacing**

The application of pavement resurfacing leads to a restoration of the original condition of the asphalt surface ($\delta_n$=1). In the case of the *Brücke Bahrenfelder Chaussee* it also reduces the pavement deterioration rate by 6% over a period of about 10 years (see 5.5.2). This activity is also assumed to result in delayed AAR-processes in the deck slab (intensity M with $\nu_{1/2}$ = 0.125/0.0625 and $dt_{1/2}$ = 1/3) as well as a reduced deterioration of the prestressing tendons in elevated position over the piers (L intensity with $\nu_{1/2}$ =0. 0625/0.03125 and $dt_{1/2}$ = 1/3)—both because of reduced water and chloride ingress (Appendix III).

The resurfacing activities require a temporary closure of at least one lane at a time, so that the availability is reduced by $1/N_{lanes}$ with $N_{lanes}$ being the total number of lanes in one driving direction. The costs are estimated according to Pommerening et al. (2008: 36) who consider a value of 11 €/m². In order to account for the ongoing deterioration of the underlying protective layer and waterproofing membrane the pavement resurfacing is limited to one application in the case of the *Köhlbrandbrücke*; after that the whole pavement must be renewed before a resurfacing can be conducted anew. However, two subsequent pavement resurfacing interventions have been conducted at the *Brücke Bahrenfelder Chaussee* according to the field data so that for this bridge a repeated application is permissible within the SAMS-model (see Appendix III). The repeated resurfacing that precedes the replacement in the case of the latter bridge is assumed to be justified by the comparatively lower percentage of heavy weight traffic running over that bridge.

**Pavement Replacement**

Similarly to the above described resurfacing, the total pavement replacement leads to a restoration of the original asphalt surface condition ($\delta_n$=1). It also leads to 40% and 20% lower pavement deterioration rates both at the eastern and western ramp of the *Köhlbrandbrücke* (see 5.5.2). Because there are no data available for the *Brücke Bahrenfelder Chaussee*, a 20% reduction is assumed for this bridge. For both bridges, the effects are assumed over a period of 20 years. Pavement replacement is also assumed to slow down the deterioration rate of prestressing tendons ($\nu$= 0. 25 for 2.5 years) and of the concrete strength in the deck slab ($\nu_{1/2}$

= 0.95/0.5 and $dt_{1/2}$ = 7/5.5). However, in contrast to mere resurfacing, the reduction of water and chloride ingress is longer lasting, since the waterproofing membrane is restored to proper condition.

The pavement replacement, like resurfacing, also reduces the availability by $1/N_{lanes}$. The costs are estimated according to Pommerening et al. (2008: 36) who consider a value of 60 €/m².

**Bridge Deck Reconstruction**

In the scope of the present thesis, only the reduction of deterioration progress is implemented into the SAMS program, while a potential improvement of the bending capacity in terms of increased height and/or concrete strength is neglected. The effect on concrete strength reduction is assumed to consist in a temporary halt ($\nu_1$=1) for a period between 7 and 9 years, which agrees with the assumed effect of a new asphalt pavement increased by an addition of 2.5 years for the water and chlorides for overcoming the new concrete layer. A further effective reduction of the strength deterioration rate in case that the newly applied concrete layer is not susceptible to AAR is neglected because the thickness of the newly applied concrete layer is considered to be small compared to the thickness of the deck slab. The reduction of the deterioration rates of the prestressing steel in the deck slab (at the high points over the piers) is assumed to consist in a temporary stop of deterioration ($\nu_1$=1) for a period of 14 to 100 years, which is in accordance with the initiation times for the medium-intensity case in Table 4–5 in chapter 4 (*sound concrete with moderately high exposition to* $Cl^-$). The effects on pavement deterioration are believed to be the same as for the above described pavement reconstruction works, simply because deck replacement always implies pavement reconstruction.

This activity is assumed to cause major traffic disruptions of 25% in case of the *Köhlbrandbrücke* and 50% in the case of the *Brücke Bahrenfelder Chaussee* during the construction works. In the case of the *Köhlbrandbrücke*, the costs for bridge deck reconstruction are derived from field data, while in the case of the *Brücke Bahrenfelder Chaussee* the costs had to be estimated (see Appendix III).

**Drainage System Overhaul**

This activity puts the condition of the drainage system back to as-new condition ($\delta_n$=1). In addition, it also reduces the deterioration rate at those locations where defects are caused by a deficient drainage system.

Accordingly, the assumption is that drainage system overhaul reduces the corrosion rate at the tendon low points and also—to a lower extent—the corrosion rate of the shear reinforcement as well as the rate of AAR-induced concrete strength reduction at the webs and the bottom slab. The estimated reduction factors $\nu_1/\nu_2$ for tendon corrosion are estimated to be around 0.5/0.125 for periods of about 3 and 5 years, while the low-intensity reduction factors are assumed to be around 0.375 and 0.125 over equal time periods. The detailed reduction factors that are applied for the example investigation are listed in Appendix III.

The overhaul of the drainage system is assumed to cause no significant traffic disruption because both case study bridges are of the box girder type and are equipped with internal pipes. In the case of the *Köhlbrandbrücke*, the costs are assumed on the basis of a cost estimate by the bridge owning authority, while in the case of the *Brücke Bahrenfelder Chaussee* the cost are estimated according to König et al. (1986: 250–282), who report value of 50 DM/m² on a time basis of 1985, and assuming a 90% surcharge because of the relative small bridge size.

**Expansion Joint Repair and Replacement**

The maintenance effects on the expansion joints themselves are estimated on the basis of field data (see 5.5.2). However, for the two bridges no field data are available for estimating the maintenance effects with respect to concrete deterioration and steel corrosion in the vicinity of leaking expansion joints. Therefore, the latter effects are estimated under consideration of the data estimates by Denton (2004: 47), who considers a reduction in the deterioration rates by 50% for a period ranging from 5 and 10 years during which the expansion joints do not leak. Keeping this in mind, replacement is assumed to be effective for around 10 years, while repair reduced the deterioration rates for around 5 years.

The replacement of the expansion joints causes considerable traffic disruptions which are estimated in the scope of this thesis to 50% for one week per joint. The costs for joint replacement and repair are estimated on the basis of field data, because those activities have been applied to both bridges.

The detailed maintenance parameter values that are used in the scope of the present investigation—referring to the physical effects, the traffic impact as well as the economic impact—are listed in Appendix III.

**Bearing Repair and Replacement**

While the maintenance effects of repair and replacement on the bearings of the *Köhlbrandbrücke* can be estimated on the basis of field data, the maintenance effect for the *Brücke Bahrenfelder Chaussee*, because there are no observations available, is assumed to consist only in a reset to a condition index value of 1.0 (sub-section 5.5.2). Further effects on elements other than the bearings themselves are not considered in the scope of this thesis.

Bearing replacement or repair is assumed to cause traffic disruptions which are estimated in the scope of this thesis to be 50% in the case of the *Köhlbrandbrücke* over periods of two days to two weeks depending of the number of bearings that are maintained, or 12.5% during one week in the case of the *Brücke Bahrenfelder Chaussee*. The costs for bearing replacement are estimated in the case of the *Brücke Bahrenfelder Chaussee* according to Pommerening et al. (2008: 36), who consider 2250 €/bearing. The costs for repair are estimated to approximately 50% of the costs for replacement. In the case of the *Köhlbrandbrücke*, the bearing maintenance costs can be estimated on the basis of available data from actually applied maintenance works. (Appendix III).

### Local Tendon and Shear Reinforcement Conservation

The activities *tendon conservation* and *shear reinforcement conservation* have local effects in terms of a temporary reduction of the corrosion rate.

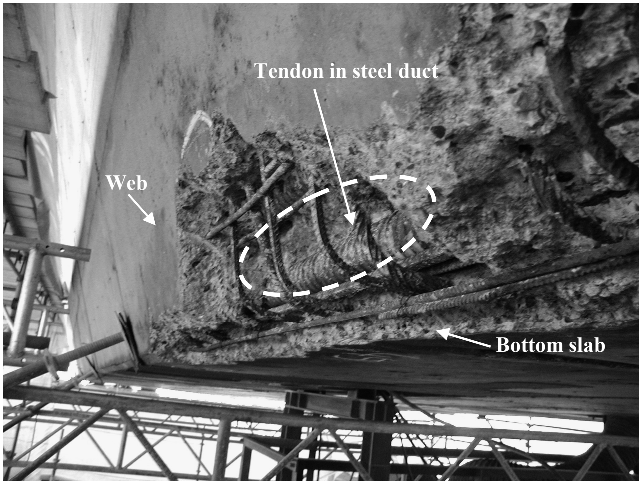



**Fig. 5.19:** Local tendon conservation: Exposed tendon in steel duct before sealing with protective mortar (Photo: Viktor Sigrist)

This measure has only a local effect in terms of a temporary halt to corrosion for around 8 years followed by a temporary 50% reduction of the corrosion rate for around 4 years. In the case of tendon conservation, the period of a standstill to corrosion is assumed to be longer than in the case of shear reinforcement conservation because the theoretically higher concrete cover of tendons—when compared to the cover of shear reinforcement—provides better conditions for conservation works.

If truck-mounted platforms or special scaffolding are used for tendon conservation at the outside of the bridge, this activity causes traffic disruptions which are estimated in the scope of this thesis to 10% (tendon) to 25% (shear reinforcement) over a period ranging from one week for the small *Brücke Bahrenfelder Chaussee* to five months in the case of the 1.500 m long part of the eastern ramp of the *Köhlbrandbrücke*. The costs are estimated in the scope of the present thesis to be in the range between 10 k€ to 50 k€ per span and pier at price levels of 2008. The lower costs are assumed for the *Köhlbrandbrücke*, because the larger bridge should allow for cheaper prices due to economies of scale. (Appendix III)

**Bending and Shear Resistance Improvement**

Both bending and shear resistance improvement is considered in the form of external FRP-strengthening. The strengthening effects are already implicitly included in the general formulation regarding the structural safety, as can be seen in sub-section 4.3.3. The equations 4.5 to 4.8, which describe the bending resistance $M_{Rd}$, include the contribution of FRP-strengthening on the cross-sectional resistance by considering the area $A_{st}$ and the ultimate stress $\sigma_{st}$ in the strengthening component. Also the expression for shear resistance $V_{Rd,s}$ in the form of equation 4.16 includes the strengthening effects in the form of $a_{st}$ and $\sigma_{st}$. Thus, strengthening is simply defined in terms of setting the values for the amount of FRP-strengthening material $\delta_{ast/Ast}$ from the initial value zero to a post-intervention value of one. The total amount of strengthening material applied at a representative cross section— $A_{st}$ and $a_{st}$—is estimated in the scope of the present investigation on the basis of the cross-sectional dimension. Further strengthening effects, such as optional web-widening by shotcrete are not considered in the scope of the present investigation. (Appendix III)

Improvement of the resistance to sagging bending action as well as shear strengthening are assumed to cause traffic flow reductions by 25% mainly by using suspended platforms for the strengthening works. Such platforms become necessary for the *Köhlbrandbrücke* because of its considerable height above ground. In the case of the *Brücke Bahrenfelder Chaussee* the disruption is assumed to be only 10% because due to its small size, truck mounted platforms with possibly lower traffic reductions may be used. Hogging bending resistance improvement, in contrast, is assumed to cause only a 10% traffic reduction because the *FRP*-strengthening is applied at the deck slab soffit from within the box-girder. The costs are estimated at 2008 prices to amount to 100 k€ per span for bending resistance improvement and to 11 k€ (*Köhlbrandbrücke*) or 12.5 k€ (*Bahrenfelder Chaussee*) per web for shear resistance improvement. (Appendix III). In order to avoid that the bridge can be infinitely be healed by adding external FRP-strengthening, these measures are limited to two applications at the same location.

**Bridge Replacement**

The economic effects of bridge replacement are included in the calculation of $C_{0,owner}$ in terms of the parameter $C_{rep}$ (sub-section 4.7.4, equation 4.59). In the scope of the present thesis, the parameter value is estimated to 110 % of the original construction costs—with price basis at the year of construction of the original bridge—in order to account for demolition and traffic management costs during bridge replacement. The construction costs for the prestressed concrete ramps of *Köhlbrandbrücke* are assumed to amount to 85 million DM in 1974, while the construction costs for the *Brücke Bahrenfelder Chaussee* could be derived from the final bill for bridge construction in 1970, which amounted to 2.53 million DM. The traffic effects are estimated to amount to traffic flow reduction of 50% ($\nu_{traf}$ = 0.5), which lasts for three years for both bridges.

## 5.6 Maintenance Strategies

### 5.6.1 Overview

The present section defines the maintenance strategies that are considered in the scope of the investigation, each of which is implementing a characteristic set of maintenance application rules in the form of a *Matlab*® code.

According to the definition in sub-section 2.6.2, a maintenance strategy describes the underlying concept of the object-oriented, holistic and long-term-oriented selection, combination and application of maintenance measures. In the scope of the present case study investigation, such an underlying concept is expressed by maintenance application rules that are specified in terms of *specific maintenance application criteria*, *combination rules*, and *time based maintenance plans*.

First, the *specific maintenance application criteria* are defined in terms of specific condition thresholds whose violation triggers the application of specifically defined maintenance activities. Here, the thresholds refer to the safety-relevant degrees of fulfilment, both with respect to bending $\eta_M$ and shear $\eta_{V+T}$, as well as to the condition of the concrete surface $CI_{CS}$, the condition of the asphalt pavement $CI_A$, and the condition of the expansion joints $CI_{Exp}$ as well as that of the bearings $CI_{Bear}$, and finally the condition of the drainage system $CI_{Drain}$. The condition thresholds are numerically defined both locally, for example as the minimum allowable $CI_{CS}$-value for a single SAMS-element, and globally, such as in the form of the overall concrete surface condition with respect to the entire bridge ($CI_{CS,res}$).

Second, the *combination rules* deal with the systematic combination of maintenance actions, such as proactive expansion joint replacement together with pavement replacement whenever the latter activity becomes necessary.

Third, the *time based maintenance plans* are defined for specific strategies and may refer to the cyclic application of hydrophobic impregnation and concrete coating as well as intensified routine maintenance application.

### 5.6.2 Definition of Maintenance Strategies in the Scope of the Investigation

Altogether eight different strategies are investigated for each case study bridge. Four of these strategies represent basic strategy forms as they are defined in sub-section 2.6.3 and Table 2.3: *reactive repair*, *proactive repair*, *preventive repair*, and *preventive-proactive*. The two basic strategy forms *full preventive* and *do nothing & rebuild* are not considered because preliminary investigations revealed clearly insufficient results for both case study bridges. In addition, the case studies investigate three different versions of *selective strategies* in order to account for the multi-objective maintenance planning process that aims at finding maintenance strategies whose performance profiles highly correspond to the bridge specific preference profiles. These three selective strategies focus on the aspects *traffic performance*, *visual* *appearance,* and *structural condition*. Also, the really applied maintenance strategies of the two case study bridges are replicated as reference strategies: These *observed strategies* result in maintenance schedules that have actually been applied in reality according to the *bridge maintenance history* as it is reconstructed from the entries in the maintenance log and from the inspection reports (HPA 2006; HPA 2010, Strom- und Hafenbau 1984, 1987a, LSBG 2010).

In the following, the eight investigated strategies are described in detail.

**Strategy $S_1$ – Reactive Repair Strategy**

The *reactive repair strategy $S_1$* reacts to critical performance developments by exclusively conducting urgent repair and strengthening activities. By doing so, activities are applied as late as possible only when an intolerable performance level is reached; besides that, no further application criteria, combination rules or time-based maintenance plans are considered.

With respect to structural safety, such an intolerable performance level occurs when the degree of fulfilment $\eta$ at any of the SAMS elements becomes insufficient. In the case of the *Brücke Bahrenfelder Chaussee*, this is defined by threshold values of $\eta$=1.0, while in the case of the Köhlbrandbrücke, a threshold value of $\eta$=0.98 is applied. The lower threshold value for the Köhlbrandbrücke is used because a value of $\eta$=1.0 for bending occurs already at the beginning at t=0 in span between axis 101 and 102 (see Sigrist & Krahwinkel 2008: 68). With respect to bridge equipment, a critical performance level occurs when the condition of at least one bridge equipment part, for example the expansion joints or the asphalt pavement, at any of the representative SAMS-elements reaches a value of zero. It is also seen as critical, when the condition of the concrete surface of any of the SAMS-element becomes zero.

In a strictly reactive sense, only those maintenance activities are conducted that are necessary to raise the most critical indicators, for example the critical expansion joint condition, to a sufficient level. Referring to the example of the expansion joints, only the critically deteriorated joints are repaired or replaced, while the other joints along the bridge, even if only slightly less deteriorated, are not subject to immediate intervention. Naturally, this strategy also does not include preventive activities, such as hydrophobic impregnation or intensified routine maintenance.

In detail, the *reactive repair strategy $S_1$* is characterised by the following maintenance application rules:

- Bending resistance improvement is applied together with local tendon conservation at the critical SAMS-element, where a value $\eta_M$=1.00 (0.98 for the *Köhlbrandbrücke*) occurs.
- Shear resistance improvement is applied together with local shear reinforcement conservation at the critical SAMS-element, where a value $\eta_{V+T}$=1.00 (0.98 for the *Köhlbrandbrücke*) occurs.

- Concrete repair works are carried out at the entire western or eastern ramp of the *Köhlbrandbrücke*, depending on whether the value $CI_{cs}$=0 occurs at an element of the western or eastern ramp. Because the *Brücke Bahrenfelder Chaussee* is comparatively small, no distinction is made between the two spans and concrete repair is always applied to the complete bridge. However, as concrete repair exclusively affects the webs, the bottom slab, the deck cantilevers, and—in the case of the Brücke Bahrenfelder Chaussee—the piers and abutments, it is not applied when the condition of the sidewalks becomes critical.
- Sidewalk rehabilitation, in analogy to the above described concrete repair application rules, this activity is applied ramp-wise in the case of the *Köhlbrandbrücke* depending on whether the value $CI_{CS,kerb}$=0 occurs at an element of the western or eastern ramp; in the case of the *Brücke Bahrenfelder Chaussee* it is applied to the entire bridge.
- Pavement replacement, in the case of the *Köhlbrandbrücke*, is also conducted at the entire critical ramp, being the one in which the SAMS-element that reaches $CI_A$=0.0 is located. In the case of the bridge *Bahrenfelder Chaussee* the complete asphalt pavement is going to be replaced when a value of $CI_A$=0.0 occurs.
- Expansion joint repair is applied exclusively at the SAMS-Element where a value $CI_{Exp}$=0.0 occurs. Alternatively expansion joint replacement is applied instead when the joint in question has already been repaired twice after the last replacement.
- Bearing repair is also applied only locally at the SAMS-Element where a value $CI_{Bear}$=0.0 occurs. Alternatively bearing replacement is applied instead when the bearing in question has already been repaired twice after the last replacement.
- Drainage system overhaul of the complete bridge is initiated when a condition value of $CI_{Drain,res}$=0.0 occurs.

These maintenance application rules ensure that the minimum requirements with respect to structural safety (shear and bending), to the bridge equipment condition, and to the concrete surface are continuously fulfilled. Consequently, this set of rules also constitutes the core of all the other investigated strategies, which are described in the following.

**Strategy $S_2$ – Proactive Repair Strategy**

The *proactive repair strategy $S_2$* aims both at maintaining a continuously raised performance level and at precluding major interventions by applying repair and strengthening timely before an actual or imminent violation of performance requirements occurs. This is achieved in the scope of the present investigation by a condition-based application of maintenance activities at deterioration levels that are less severe than the critical level *CI*=0. Also, certain combination rules for coordinated maintenance action are applied. However, in its theoretical pure form it does not include special preventive activities of any kind, as opposed to the below described preventive-proactive strategy $S_4$.

In addition to the basic core of application rules, which is defined by the above described strategy $S_1$, the following proactive maintenance application rules are central for the proactive repair strategy $S_2$:

- Tendon conservation is proactively applied along the entire bridge in combination with essential local bending resistance improvement. In contrast to local strengthening, which is applied exclusively to the critical SAMS-element, tendon conservation works are conducted proactively to the entire bridge, regardless of the local deterioration levels. Additionally, tendon conservation is proactively applied at all span elements whenever concrete repair works are conducted.
- Shear reinforcement conservation is also applied proactively following the above described procedure for tendon conservation.
- Concrete repair works are carried out along the complete bridge when an overall concrete surface condition of $CI_{CS,res}$=0.375 is reached. In addition, the concrete works also include sidewalk rehabilitation when the latest sidewalk rehabilitation works date back more than ten years. According to the rating scale that is introduced in Appendix II, a rating value of $CI_{CS,res}$=0.375 describes a condition halfway between fair and poor, where cracking, breaking, and leakages are still mostly local issues, but already are at the verge of spreading across the whole bridge.
- Pavement resurfacing is applied along the complete bridge whenever an overall pavement condition of $CI_{A,res}$=0.375 is reached. Alternatively, pavement replacement is applied when resurfacing has already been applied more than once in the case of the *Köhlbrandbrücke* or more than twice in the case of the *Brücke Bahrenfelder Chaussee* (for the latter special case, see sub-section 5.5.3). The threshold rating of $CI_{A,res}$ =0.375 describes a condition halfway between fair and poor, which already imposes adverse effects on traffic flow and is characterised by a worn down road surface, considerable surface deformations, as well as potholes and cracking (see Appendix II).
- Expansion joint repair is carried out exclusively at the SAMS-Element where a value $CI_{Exp}$=0.375 occurs. In addition, all expansion joints whose last repair or replacement dates back more than four years are proactively repaired whenever pavement repair or replacement works are conducted. In both cases, expansion joint replacement is applied instead for those joints that have already been repaired twice after their last replacement. The ‘fair to poor’ condition at a rating of $CI_{Exp}$=0.375 is characterised by defects such as heavy corrosion, loose or broken anchorages and leakages (see Appendix II).

- Bearing repair is also exclusively applied at the SAMS-Element where a bearing condition of $CI_{Bear}$=0.375 occurs. Alternatively, bearing replacement is carried out instead when the bearing in question has already been repaired twice after the last replacement. The 'fair to poor' condition at a rating of 0.375 is characterised by heavier defects, which however do not yet pose a danger to structural safety (see Appendix II).
- Drainage system overhaul of the complete bridge is initiated when a value $CI_{Drain,res}$=0.375 occurs. The 'fair to poor' condition at a rating of 0.375 is characterised by defects, which already have adverse effects on the function of the drainage system and which already are becoming a general issue (see Appendix II).

**Strategy $S_3$ – Preventive Repair**

The *preventive repair strategy* aims at postponing repair and strengthening interventions by applying preventive activities. It combines the reactive repair approach of the above described strategy $S_1$ with cyclic hydrophobic impregnation. Moreover, intensive routine minor maintenance is regularly applied.

Thus, in addition to the basic core of application rules, which is defined by the reactive strategy $S_1$, the following preventive application rules are central for the preventive repair strategy $S_3$:

- Intensified routine minor maintenance is carried out on a regular base.
- Hydrophobic impregnation of the bridge girders is routinely applied in ten-year intervals. In the case of the *Brücke Bahrenfelder Chaussee*, impregnation works also refer to the piers and abutments. In the case that the cyclic application was suspended because of insufficient concrete surface conditions, a new ten-year-cycle starts after essential concrete repair works have been applied. As described in sub-section 5.5.3, insufficient surface conditions are assumed when the local condition $CI_{cs}$ of the webs, or the bottom slab, or the deck cantilevers (or the piers and abutments) falls below a value of 0.375. In the case of the *Köhlbrandbrücke*, girder impregnation of the eastern and western ramp is applied individually, so that insufficient surface conditions on one ramp do not affect treatment of the other ramp. For the *Brücke Bahrenfelder Chaussee* only complete treatment is considered. For both bridges, it is independently applied from kerb and sidewalk impregnation.
- Hydrophobic impregnation of the kerbs and sidewalks is routinely applied in ten-year intervals analogously to the above described girder impregnation. It is also suspended when the sidewalk concrete condition $CI_{CS,kerb}$ falls below a value of 0.375. As it is the case with girder impregnation, it is individually applied to the eastern and western ramp of the *Köhlbrandbrücke* and along the entire *Brücke Bahrenfelder Chaussee*, respectively.

**Strategy $S_4$ – Preventive-Proactive Strategy**

The *preventive-proactive strategy* combines proactive repair interventions at still uncritical deterioration levels with special preventive activities and intensified routine maintenance. Essentially, the objective of this strategy is similar to the preventive repair strategy, in that it aims at postponing—or even precluding—repair and strengthening interventions by preventive activities. But in terms of the additional proactive aspect, it also aims at maintaining a continuously good performance by early maintenance application. In the scope of the present SAMS investigation the preventive-proactive repair strategy $S_4$ is simply defined as the proactive repair strategy $S_2$ with additional preventive hydrophobic coating and intensified routine maintenance.

Consequently, this strategy is characterised by three complementary sets of maintenance application rules. The first set comprises the core application rules of strategy $S_1$, the second set includes the proactive maintenance application rules of strategy $S_2$, and finally the third set of rules appears in the form of the preventive application rules that are defined for the strategy $S_3$.

**Strategy $S_{5.1}$ – Traffic Performance**

This strategy aims at providing a bridge that—in its role as a traffic link—shows high levels of traffic performance. It puts emphasis on a high average condition of the asphalt pavement and of the expansion joints because both bridge equipment elements directly determine the traffic performance of the bridge. More specifically, this strategy follows a proactive approach to pavement and expansion joint maintenance to keep them above 'fair' to 'good' condition which according to equations 4.67 and 4.68 results in a only slightly reduced traffic performance of around 93% of the initial capacity. As a supporting measure, intensified routine maintenance also shall improve the average traffic performance. The other elements, such as the bearings or the concrete surface, are merely reactively maintained. Also, intensified routine minor maintenance is applied.

In detail, the traffic-flow centred strategy $S_{5.1}$ comprises the following <u>special application rules</u> in addition to the core of rules that constitute the basic strategy $S_1$:

- <u>Intensified routine minor maintenance</u> is carried out on a regular base.
- Proactive <u>pavement resurfacing</u> is applied along the complete bridge when an overall pavement condition of $CI_{A,res}$=0.625 is reached. Alternatively, <u>pavement replacement</u> is applied when resurfacing has already been applied more than once in the case of the *Köhlbrandbrücke* or more than twice in the case of the *Brücke Bahrenfelder Chaussee*. The threshold rating of $CI_{A,res}$ =0.625 describes a condition halfway between 'good' and 'fair', where first adverse effects on traffic flow occur (see equations 4.67 and 4.68). The typical condition at this stage is described by medium surface defects, including ravelling and pol-

ishing, local cracking, rutting with a depth of around 2 cm, heavings of around 3cm, and potholes with a depth of up to 3 cm (see Appendix II).

- Proactive expansion joint repair is carried out to all joints along the entire bridge when the representative condition $CI_{Exp,res}$ reaches a value of 0.625. In addition, all expansion joints whose last repair or replacement dates back more than four years are proactively repaired whenever pavement repair or replacement works are conducted. In both cases, expansion joint replacement is applied instead for those joints that have already been repaired twice after their last replacement. The ‘good’ and ‘fair’ condition at a rating of $CI_{Exp}$=0.625, which already causes some adverse effects on traffic flow (see equations 4.67 and 4.68), is characterised by minor to moderate defects such as medium corrosion, loose or damaged seals, and loosening components (see Appendix II).

**Strategy $S_{5.2}$ – Visual Appearance**

The strategy $S_{5.2}$ systematically reduces adverse influences on the visual appearance by applying intensified routine minor maintenance and cyclic concrete coating. Moreover, the expansion joints are proactively repaired or replaced because leaking joints can lead to locally visible defects to the concrete structure. Also, proactive pavement resurfacing or repair is carried out in order to maintain a barrier against water ingress. This barrier effectively reduces leakage related local damages of the deck cantilever soffit and also limits periods of below-fair appearance of the carriageway.

In detail, the strategy $S_{5.2}$ comprises the following special application rules in addition to the common core of rules that applies to all investigated strategies:

- Intensified routine minor maintenance is carried out on a regular base.
- Coating of the bridge girders as well as of the kerbs and sidewalks is carried out along the complete bridge when an overall concrete surface condition of $CI_{CS,res}$=0.625 is reached. According to the applied rating scale (Appendix II), a rating value of $CI_{CS,res}$=0.625 describes a condition halfway between ‘good’ and ‘fair’, where cracking, breaking, and leakages are mostly local issues, and adverse effects on visual appearance are still a minor issue.
- Concrete repair works are carried out in combination with sidewalk rehabilitation and coating along the complete bridge when an overall concrete surface condition of $CI_{CS,res}$=0.625 is reached and coating has been applied more than threefold before.
- Proactive pavement maintenance is applied according to the proactive strategy $S_2$ at an asphalt condition level of $CI_{A,res}$=0.375.

- Proactive expansion joint maintenance is carried out similarly to the proactive strategy $S_2$ at a condition level of $CI_{Exp}$=0.375. However, proactive joint maintenance is not applied in combination with asphalt pavement.

**Strategy $S_{5.3}$ – Structural Condition**

The strategy $S_{5.3}$ systematically extends the structural service life by reducing structural deterioration processes. A crucial aspect of this strategy, which clearly distinguishes it from the other investigated strategies, is its focus on preserving the structural safety by proactive interventions with respect to the shear and bending capacity. This is achieved by proactive conservation of shear reinforcement and prestressing tendons at subcritical performance levels, by proactive concrete repair works, and by applying proactive bridge deck reconstruction. In addition, proactive maintenance is applied to all bridge equipment elements. Furthermore, structural deterioration processes are reduced by also applying intensified routine minor maintenance and cyclic hydrophobic impregnation.

This strategy closely resembles the proactive-preventive approach of strategy $S_4$. However, there are two characteristic differences between these two strategy types.

The first difference is the stronger focus on the structural condition: Routinely, shear reinforcement and tendon conservation is applied along the whole bridge when concrete repair works are applied—independently of the actual condition of the shear or prestressed reinforcement and also independently of the actual reason for concrete rehabilitation works.

Moreover, major concrete repair works are also conducted when structural strengthening (shear or prestressing) becomes necessary. Such major concrete rehabilitation works are applied in the form of girder concrete repair or even bridge deck reconstruction works. The first measure—concrete rehabilitation—is applied when threshold violation occurred for shear or positive bending. The latter measure is applied in the case that the sub-critical threshold violation occurred for negative bending action over the pier. However, both additional major rehabilitation works are only applied, when the last concrete works of that kind lay back more than 20 years.

The second difference is that the strategy $S_{5.3}$, in comparison to the previously described strategy $S_4$, puts higher demands on the bridge equipment parts. A lot of attention, for example, is paid to the expansion joints. This is because danger to structural health is assumed to occur with leaking expansion joints, so that the threshold value is set to $CI_{Exp}$=0.5, which is defined as the condition index where—among other defects—leaks occur at the joints (see Appendix II). Similarly, cracks that extend through the asphalt pavement can lead to structural deterioration of the bridge deck and prestressing tendons. Thus, the threshold for the pavement condition $CI_A$ is set to 0.375 for the worst deteriorated SAMS-element. In an analogous way, drainage system overhaul is applied in such a way that the condition does not fall below a value of $CI_{Drain}$=0.5, which describes defects that result in adverse effects on function and structure (see Appendix II). Only the bearing condition development perfectly corresponds to the preventive-proactive strategy. This is because, as it is assumed in the scope of the present thesis, the bearing condition does not affect other bridge elements.

In detail, the strategy $S_{5.3}$ comprises the following special application rules in addition to the common core of reactive rules that applies to all investigated strategies:

- Intensified routine minor maintenance is carried out on a regular basis.
- Hydrophobic impregnation of the bridge girders as well as hydrophobic impregnation of the kerbs and sidewalks is routinely applied in ten-year intervals, following the same procedure that is described for strategy $S_3$.
- Tendon conservation is proactively applied at subcritical performance thresholds as well as together with essential local resistance improvement activities. In the case of the *Köhlbrandbrücke*, where the critical degree of fulfilment for bending ($\eta_M$) is set to 0.98, proactive tendon conservation is applied at a subcritical value of $\eta_M$=1.0 to all SAMS-elements that represent positive bending action; these are the elements where the bending capacity refers to cross-sections with tendons at their low points. This restriction accounts for the special vulnerability of tendons at their low points in box-girder cross-sections which often show poor concrete compaction around the tendons (Zilch and Weiher 2007; König et al. 1986: 200–201); such weak points were for example found to be locations of considerable corrosive tendon attack of the *Köhlbrandbrücke* (see section 5.5.2). In the case of the *Brücke Bahrenfelder Chaussee* these conservation activities are applied at a higher threshold level of $\eta_M$=1.25 because the initial degree of fulfilment is found to be $\eta_M$=1.60 which allows for such a generous intervention level. In addition, tendon conservation is proactively applied along the entire bridge in combination with concrete repair works as well as together with the essential local shear resistance improvement.
- Shear reinforcement conservation is proactively applied analogously to the above described tendon conservation procedures. The threshold levels for subcritical intervention is set to $\eta_{V+T}$=1.0 for the *Köhlbrandbrücke* and to $\eta_{V+T}$=1.25 for the *Brücke Bahrenfelder Chaussee*.
- Concrete repair works are carried out along the complete bridge when an overall concrete surface condition of $CI_{CS,res}$=0.50 is reached. According to the rating scale that is introduced in Appendix II, a rating value of $CI_{CS,res}$=0.50 describes a ‘fair’ condition with mostly locally concentrated defects, which already exert adverse effects on appearance. In addition, concrete repair works are also proactively conducted when a subcritical degree of fulfilment with respect to shear or positive bending occurs ($\eta_M$=1.00 or $\eta_M$=1.25, depending on the bridge in question), provided that the latest concrete repair works date back more than 20 years.

- Bridge deck reconstruction is also applied when a subcritical degree of fulfilment with respect to negative bending occurs a pier elements ($\eta_M$=1.00 or $\eta_M$=1.25, depending on the bridge in question), provided that the latest deck reconstruction works date back more than 20 years.
- Pavement replacement, in the case of the *Köhlbrandbrücke*, is also conducted at the entire critical ramp, being the one in which the SAMS-element that reaches $CI_A$=0.375 is located. In the case of the bridge *Bahrenfelder Chaussee* the complete asphalt pavement is going to be replaced when a value of $CI_A$=0.375 occurs.
- Expansion joint repair is applied exclusively at that SAMS-Element where a value $CI_{Exp}$=0.50 occurs; replacement is applied instead when more than two repair action have been conducted already. The stricter performance threshold compared to strategy $S_2$, which initiates action only at $CI_{Exp}$=0.375, is chosen in order to prevent heavier leakages because these can result in higher corrosion of steel and concrete. Also, as for strategy $S_2$, all expansion joints whose most recent repair or replacement dates back more than four years are proactively repaired whenever pavement repair or replacement works are conducted.
- Bearing repair, or alternatively replacement, is applied at that SAMS-Element where a bearing condition of $CI_{Bear}$=0.375 occurs according to the procedure for the proactive strategy $S_2$.
- Drainage system overhaul of the complete bridge is initiated when a value $CI_{Drain,res}$=0.50 occurs. The 'fair' condition at a $CI_{Drain,res}$ -rating of 0.50 is characterised distributed defects that exert adverse effect on function and structure.

**Strategy $S_0$ – Observed Strategy**

This strategy is tailored to replicate the actually applied maintenance schedule from the year of completion until the end of the maintenance records. In order to do so, the condition indicator thresholds, time-base maintenance plans and combination rules are defined in such a way that the results of a SAMS-simulation—where all input parameter values are set to the observed values (or most likely values)—mimic both the really applied maintenance schedule and the observed condition developments.

Beyond that time horizon, which is the case of the *Köhlbrandbrücke* 37 years and in the case of the *Brücke Bahrenfelder Chaussee* 39 years after construction, the follow-up maintenance application is further governed by the defined condition indicator thresholds, time-base maintenance plans and combination rules.

This replication of the actually applied strategies serves two purposes. First of all, it is used as a test on the accuracy of the established SAMS-model. The accuracy is judged depending on how well the simulation results for the condition development correspond to the observed data. In the present case study investigations, the accuracy test is conducted by comparing the simulation results which follow when all parameter values (deterioration and maintenance) are set to the 50%-percentile of their respective distributions, to the observed data. Second, the reproduction of the actually applied strategy sets a benchmark against which all other strategies can be evaluated.

In the case of the *Köhlbrandbrücke*, the following maintenance application rules are derived from the available bridge inspection and maintenance data:

- Special preventive activities, such as intensified routine minor maintenance or hydrophobic impregnation, are not applied.
- Tendon conservation is proactively applied at subcritical performance thresholds as well as together with concrete repair works and essential local bending resistance improvement activities. The first intervention type represents the actually applied maintenance works that were carried out after severe tendon corrosion had been detected in the spans 4/5 and 50/52 (see section 5.5.2) and also at other location along the entire bridge, where the deterioration progress was much slower than at the above mentioned spans. On the basis of a simplifying interpretation of the observed data, it is assumed that tendon conservation at SAMS-element '57.5', which represents the span 50/52, is applied 12 years after completion. The other tendon conservation works along the rest of the bridge, and thus at the remaining SAMS-elements, have been applied around the year 19 after construction together with the works in span 4/5. Further proactive tendon conservation is applied at a subcritical value of $\eta_M$=1.0 to all remaining SAMS-elements that represent positive bending action ('117.5', '101.5', and '01.5'). These are the locations which often show poor concrete compaction around the tendons (Zilch and Weiher 2007; König et al. 1986: 200–201). The second intervention type—tendon conservation works together with concrete repair works as well as with local resistance improvement activities—is implemented into the SAMS strategy definition in order to account for the general proactive attitude towards tendon conservation that is suggested by the observed maintenance practice. This rule also succeeds to replicate the observed tendon conservation works together with the major rehabilitation program which is reported to take place from 2009 to 2012.
- Shear reinforcement conservation is proactively applied on a global scale analogously to the above described tendon conservation procedures at subcritical levels and together with local essential shear resistance improvement. Notwithstanding the fact that shear reinforcement conservation or strengthening has not yet been applied in reality, this application rule shall also account for the generally observed proactive attitude to structural issues. The threshold levels for subcritical intervention is set to $\eta_{V+T}$=1.0.
- Bending resistance improvement is applied together with local tendon conservation at that critical SAMS-element, where a value $\eta_M$= 0.98 occurs. After the first strengthening, the threshold value is increased to $\eta_M$=1.0.

- Shear resistance improvement is applied together with local shear reinforcement conservation at that critical SAMS-element, where a value $\eta_{V+T}$= 0.98 occurs. After the first strengthening, the threshold value is increased to an assumed value of $\eta_M$=1.025.
- Concrete repair works are carried out at an overall concrete surface condition $CI_{CS,res}$=0.20 in combination with sidewalk rehabilitation along the complete bridge and also in combination with tendon conservation at the span-elements ('117.5', '101.5', '01.5', and '57.5'). The threshold value $CI_{CS,res}$=0.20 follows from the overall concrete condition that is found from extrapolating the bridge inspection reports of 2006 to the time of the major concrete repair works in 2010.
- Sidewalk rehabilitation is applied to the western ramp 28 years after bridge construction. After this, sidewalk rehabilitation is applied in combination with concrete repair works. It is also applied to that ramp, where the sidewalks reach a condition of 0.125; however this rule only applies after the first major concrete repair works have been conducted. This is because at the eastern ramp, the observed condition of sidewalks has been allowed to deteriorate close to zero, so that a 0.125 threshold has not been applied here. However, a value of 0.16 was observed during the deck reconstruction works in the year 16, which also included sidewalk rehabilitation. In so far, the poor condition of the eastern sidewalks can be explained in such a way that sidewalk maintenance was deliberately postponed in order to combine it with the planned concrete repair works.
- Bridge deck reconstruction is applied along the entire bridge when the resulting condition of the asphalt pavement for the first time reaches a value of $CI_{A,res}$=0.464, which is derived from the inspection and maintenance data analysis. After this application, deck reconstruction works are not being conducted. Instead, pavement replacement will be the measure of choice.
- Pavement replacement, is conducted along the entire bridge at an alternating threshold value of $CI_{A,res}$=0.464 and $CI_{A,res}$=0.375.
- Expansion joint repair is carried out to all joints along the entire bridge when the representative condition $CI_{Exp,res}$ reaches a value of 0.25. Expansion joint replacement is applied instead when the joints have already been repaired twice after their last replacement; this is done at a condition value of $CI_{Exp,res}$=0.53. Consequently, all expansion joints are subject to the following cycle: 2 × repair ($CI_{Exp,res}$=0.25) followed by 1 × replacement ($CI_{Exp,res}$=0.53). This cycle is a simplified representation of the observed data.
- Bearing repair is applied when a value of $CI_{Bear}$=0.0 occurs. The bearing repair depends on the location of the critical location: Repair is applied to the entire western ramp when $CI_{Bear}$ =0.0 occurs at an element of the western ramp ('117', '118', '101'), otherwise it is applied to the element that represent the axis range 01-30 of the eastern ramp ('01'), or it is applied to the axis range 31–42 of the eastern ramp (element '57.9'), or repair is applied to

the bearings at the pier at the stay-cable bridge (‘00’). Bearing replacement is applied instead when the bearings in question have already been repaired twice after the last replacement. The only exception to this rule is that the roller bearing must be replaced first before the repair-replacement cycle starts.

- Drainage system overhaul of the complete bridge is initiated when a condition value of $CI_{Drain,res}$=0.131 occurs.

In the case of the *Brücke Bahrenfelder Chaussee*, the following maintenance application rules replicate the actually applied strategy:

- Coating of the bridge girders as well as coating of the kerbs/sidewalks is applied together with concrete repair works and sidewalk rehabilitation works.
- Bending resistance improvement is applied together with local tendon conservation at the critical SAMS-element, where a value $\eta_M$=1.00 occurs.
- Shear resistance improvement is applied together with local shear reinforcement conservation at the critical SAMS-element, where a value $\eta_{V+T}$=1.00 occurs.
- Concrete repair works are carried out at an overall concrete surface condition $CI_{CS,res}$=0.47 in combination with sidewalk rehabilitation along the complete bridge and also in combination with coating of the girders together with the piers / abutments and coating of the kerbs/sidewalks.
- Pavement resurfacing is applied along the complete bridge when an overall pavement condition of $CI_{A,res}$=0.25 is reached. Alternatively, pavement replacement is applied when resurfacing has already been applied more than twice. Other than pavement resurfacing, which actually took place 18 and 36 years after bridge construction, pavement replacement has not been applied so far. However, it is assumed in the scope of the present investigation that after repeated resurfacing only a replacement can restore a sufficient condition.
- Expansion joint replacement is carried out when the condition value $CI_{Exp,res}$ becomes 0.25. Following this intervention, expansion joint repair is conducted when the $CI_{Exp,res}$ reaches a value of 0.75. Thereafter, replacement is applied again. Consequently, the expansion joint of the bridge is subject to the following cycle: 1 × replacement ($CI_{Exp,res}$=0.25) followed by 1 × repair ($CI_{Exp,res}$=0.75).
- Bearing repair has not been conducted so far. Therefore a basic reactive approach is assumed for the bearing maintenance regime: All bearings are repaired when a value $CI_{Bear}$=0.0 occurs. Bearing replacement is applied instead when the bearings have been repaired twice after the last replacement.
- Drainage system overhaul also has not been conducted so far. It is assumed that drainage system overhaul is initiated when a condition value of $CI_{Drain,res}$=0.0 occurs.

## 5.7 Simulation-based Strategy Assessment

### 5.7.1 Stochastic Sample and Illustrative Data Combination

A sample of 100 randomly generated value combinations for the deterioration and maintenance parameters is used for strategy simulation. This sample is generated by using crude Monte Carlo simulation technique on the basis of triangular distribution functions. All parameter distributions are treated as being statistically independent from each other within the given value ranges that depend on the intensity level (H, M, L). This sample of size N=100 constitutes the data basis for strategy assessment and further evaluation.

Each strategy simulation is conducted for the identical sample of 100 elements, which are processed in identical order so that the alternative strategies can be compared under identical experimental conditions. Consequently the observed differences in performance can be attributed with confidence to the strategy specific maintenance application rules (cp. Law 2008: 579). For example, the service life $T_{SL}$ that results from application of maintenance strategy $S_1$ on the specific parameter-value combination of sample element No. *xy*—i.e. corrosion rates, maintenance effects, etc—can be directly compared to the $T_{SL}$-value that results for strategy $S_4$ for the same sample element No. *xy*.

In addition to this stochastic sample, a single *illustrative data combination* is introduced, which represents the 50$^{th}$ percentile of the deterioration and maintenance parameters. This data combination is used for illustrative purposes, in particular for indicating that the SAMS-model is sufficiently adjusted to the field data and also for illustrating the characteristic condition development picture for each investigated strategy, which otherwise is blurred by the overlapping curves for the 100 single sample elements.

### 5.7.2 Strategy $S_0$ – Real Strategy

#### Overall Condition Development

It can be seen that the overall condition development for both bridges—as it results from SAMS-based strategy simulation—is consistent with the condition development found by analysis of the data provided by inspection and maintenance records (Fig. 5.20, Fig. 5.21). In both cases, the condition development that results from SAMS-simulation of the illustrative data combination, which is the combination of the 50$^{th}$ percentiles of each of the maintenance and deterioration parameters, reproduces the actually observed development with a very high accuracy. The really applied maintenance schedule is correctly replicated by the simulation results in the form of the application times of each maintenance intervention. Moreover, the condition development for the bridge equipment parts and the concrete surface, as it is reconstructed from the field data, is almost perfectly repeated by the simulation results. Consequently, the SAMS model is sufficiently adjusted to the field data and thus can be expected to yield reliable results for the assessment of the actually applied as well as of alternative maintenance strategies.

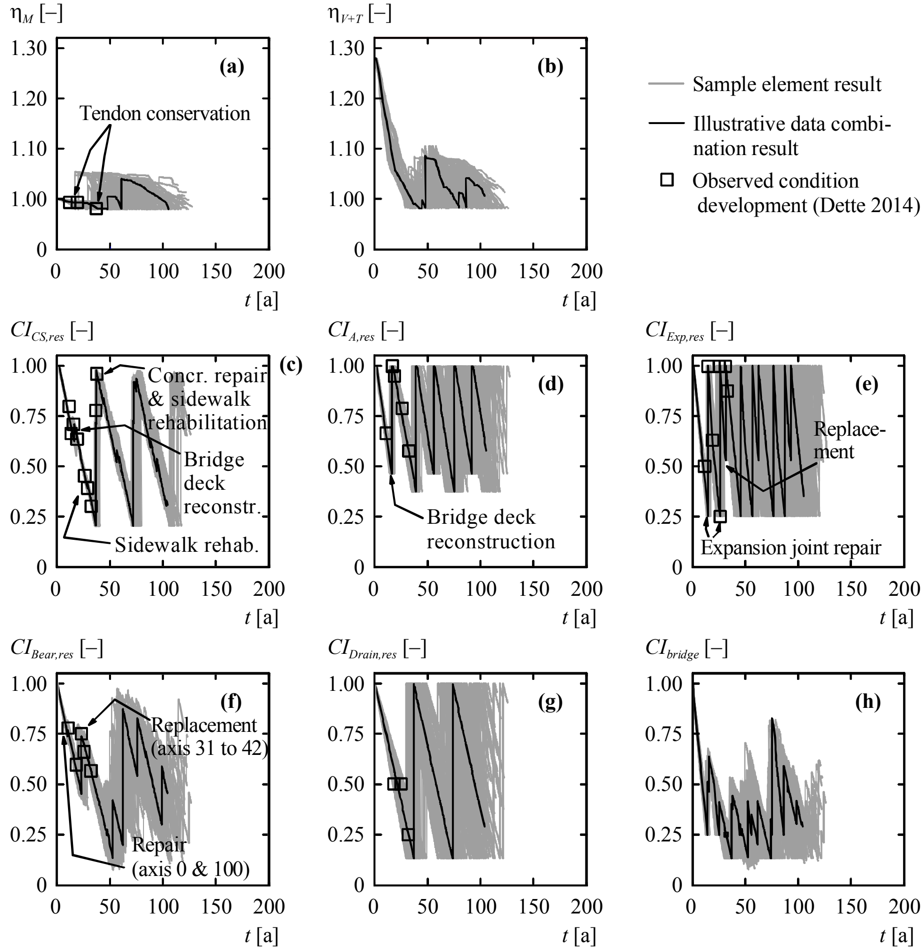



**Fig. 5.20:** Strategy $S_0$ (Köhlbrandbrücke): Simulation results for each one of the $N$=100 sample elements (grey lines) and the illustrative data combination (black line) compared to the observed condition development (scatter points).

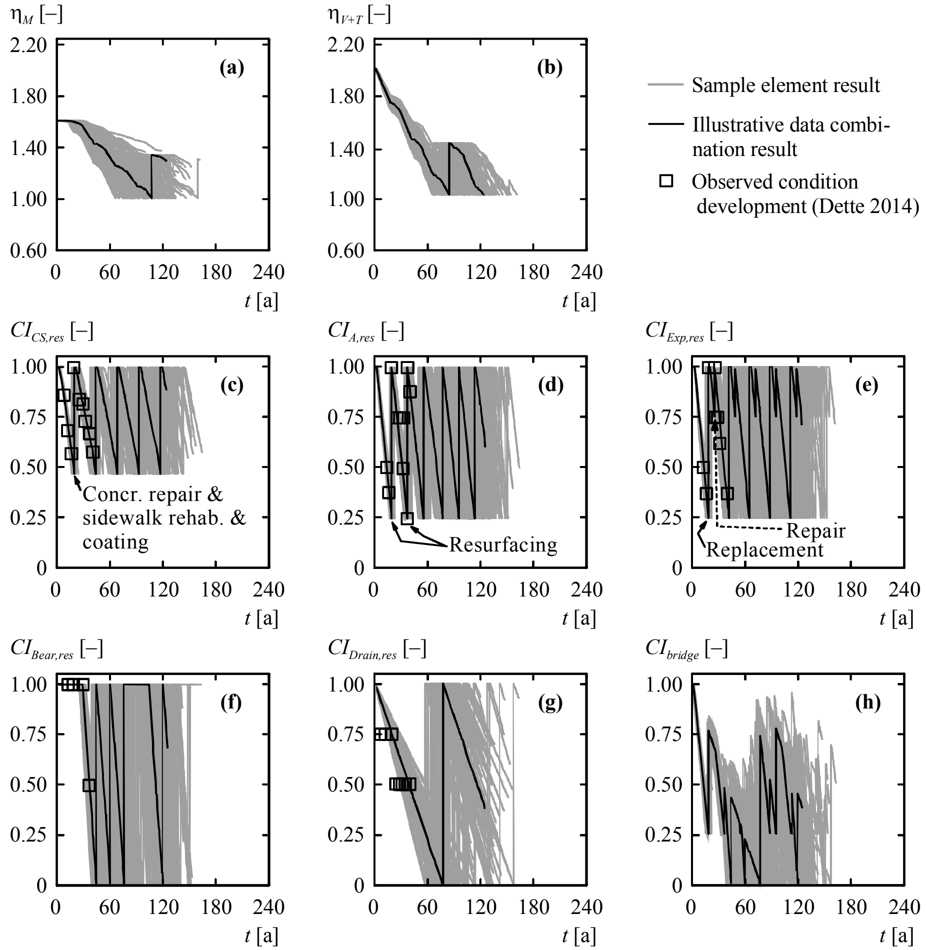



**Fig. 5.21:** Strategy $S_0$ (Bahrenfelder Chaussee): Simulation results for each one of the $N$=100 sample elements (grey lines) and the illustrative data combination (black line) compared to the observed condition development (scatter points).

**Development of the Structural Safety**

The development of the structural safety is expressed in terms of the degrees of fulfilment $\eta_M$ and $\eta_{V+T}$. However, it must be noted, that the developments of the shear and bending capacities in the scope of the present investigation are purely illustrative. In contrast to the condition development of the bridge equipment and of the concrete surface, which are based on observation data, no reliable information on reinforcement corrosion and concrete strength reduction are at hand; the data referring to the corrosion progress at span 4/5 and 50/52 of the *Köhlbrandbrücke* are only of indicative character for the entire bridge. The same applies to the concrete strength test referred to in Fig. 5.12. In the case of the Brücke Bahrenfelder Chaussee there are no data available at all. Moreover, the cross section properties, which are used for calculation of the cross-sectional resistances of both case study bridges, are only of representative nature with simplified geometries and partially adapted amounts of steel reinforcement. Also, the structural model of the *Brücke Bahrenfelder Chaussee* is a simplified beam model with simplified tendon geometry and a reduced set of investigated load cases. In summary, the simulation results regarding the aspects of structural safety, as they are discussed in the following for the illustrative data combination, are only of indicative nature.

In the case of the *Köhlbrandbrücke*, the tendon conservation works, which are applied in the year 12 and 18 and together with the concrete repair works in the years 36 and 37, cause a noticeable temporary halt in the deterioration progress of the structural safety with respect to bending (see Fig. 5.22 (a)). However, because of the initially low degree of fulfilment $\eta_M$=1.00, which does not leave much reserve to the critical level of $\eta_M$=0.98, the bending capacity becomes the service life limiting factor. In the case of the illustrative data combination, only a service life of 104 years can be achieved, which necessitates first strengthening works 46 years after construction at the element '101.5' and one more strengthening intervention after 60 years at element '01.5' (Fig. 5.22 (a)). Also, extensive shear strengthening becomes necessary around 50 years after construction at the SAMS-elements '117', '57.9', and '101' (see Fig. 5.22 (b)). However, shear does not constitute the lifetime limiting factor, even if two more essential strengthening interventions are applied at element '00' and '01' at a time of 78 and 86 years after construction respectively. The remarkably low strengthening effect that occurs at axis '101', as it can be seen in Fig. 5.22 (b) by comparing it to strengthening effects at axis '00' or '01', results from the fact that axis '101' represents the location with the highest shear and therefore the highest shear reinforcement ratio along the whole bridge. Consequently, the relative strengthening effect of additional FRP-sheets at axis '101' is lower compared to the other locations. It should be mentioned that in the scope of the present investigation, at axis '101', extensive FRP strengthening with twice the amount at the other locations is considered. It also should be noted that the strikingly steep decrease in the shear capacity, as it occurs immediately after strengthening at the elements '57.9', '117', and '00', is due to the initially high deterioration of the FRP-strengthening systems over the first years as it is discussed in section 5.5.2. In contrast, the decrease in shear capacity at element '101' after strengthening is not that steep because of the comparatively lesser strengthening level compared to the existing reinforcement ratio, so that the initially fast decay of the FRP does effect a lesser share of the overall reinforcement, which then consists of steel and FRP. In the case of the *Brücke Bahrenfelder Chaussee*, in contrast, shear becomes the lifetime limiting factor after 125 years, with one preceding shear strengthening intervention at the element '02' in the year 84 after construction (Fig. 5.22 (d)). Only one bending resistance improvement becomes necessary in span element '0.5' around 107 years after construction (Fig. 5.22 (c)). The temporarily reduced deterioration rates of the shear and bending capacities, as they appear in the wavy deterioration curves in Fig. 5.22 (c) and (d), result from the concrete coating, which is applied together with the concrete repair works. The time span with a nearly constant degree of fulfilment with respect to bending over the first 20 years of the service life is due to the low deterioration rate of the tendons over the pier at element '01M', which are protected by a considerable concrete cover and the pavement. The sudden increase of the $\eta_{M,res}$-degradation occurs when the faster deterioration curve belonging to the element '0.5' crosses the '01M'-curve (Fig. 5.22 (c)).

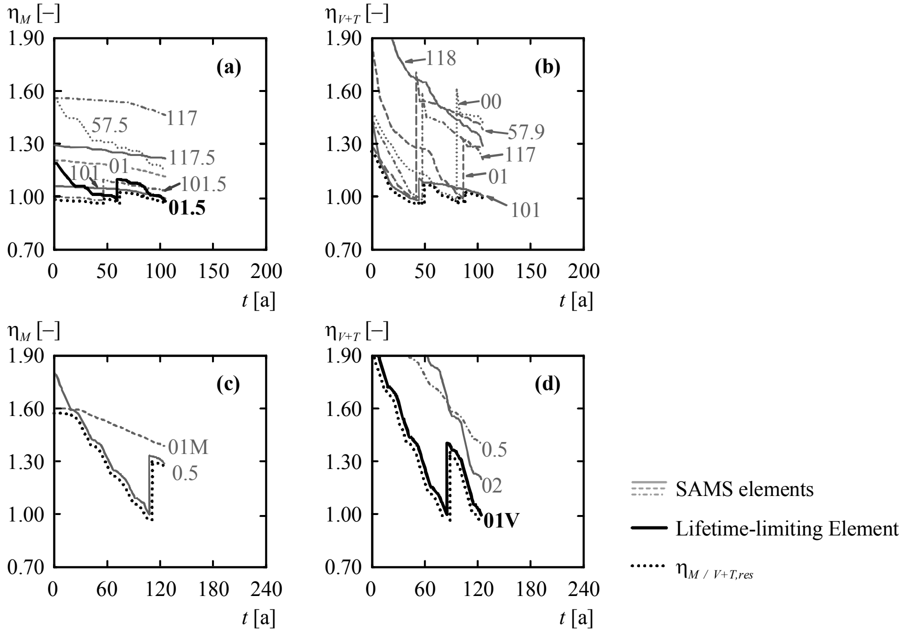



**Fig. 5.22:** Structural Safety: Time-dependent development of the degrees of fulfilment $\eta_M$ and $\eta_{V+T}$ for the Köhlbrandbrücke ((a) and (b)) and for the Brücke Bahrenfelder Chaussee ((c) and (d)) in the case of the illustrative data combination.

**Development of the Concrete Surface Condition**

On the macro-scale, as it appears in Fig. 5.20 (c) and Fig. 5.21 (c), both bridges show a generally similar concrete surface condition development in the form of a cyclic alternation of $CI_{CS,res}$ between a very good condition and a lower threshold value, which is 0.2 in the case of the *Köhlbrandbrücke* and 0.47 in the case of the *Brücke Bahrenfelder Chaussee*.

However, a closer look reveals an elementary difference: In the case of the *Brücke Bahrenfelder Chaussee*, sidewalk rehabilitation works are always applied together with concrete repair works, while in the case of the *Köhlbrandbrücke* sidewalk rehabilitation is individually applied, which causes the repeated small jumps in the $CI_{CS,res}$-curve.

In the case of the *Brücke Bahrenfelder Chaussee*, the kerbs/sidewalks in fact are very small edge beams along the deck cantilevers, and thus the combination of girder and sidewalk concrete repair is the natural decision. This combination practice is applied during the observed major concrete repair 18 years after completion, which included girders, abutments and the kerbs/sidewalks. The fact that the subsequent concrete repair occurs at a higher condition level for the sidewalks and the webs as well as the abutments compared to the condition at the first observed intervention simply results from the fact that the preventive coating is found to better affect these parts of the bridge compared to the bottom slab and the deck cantilevers.

In contrast, the sidewalks of the *Köhlbrandbrücke* represent a bigger share of the concrete structure so that their maintenance can reasonably be applied independently from girder concrete repair—which is indeed the observed intervention practice. From the bridge inspection data, intervention thresholds are found to be $CI_{CS,res}$=0.20 for girder concrete repair along both ramps and $CI_{CS,kerb}$=0.125 for ramp-specific kerb/sidewalk rehabilitation. The latter rule is a rather rough representation of the observed maintenance application practice with its quite irregular pattern: While the first sidewalk rehabilitation is observed at a condition of 0.13 in combination with the bridge deck rehabilitation works, the second intervention along the western ramp is applied at a sidewalk condition of around 0.25 and at the eastern ramp at a condition of close to zero in combination with girder concrete repair works.

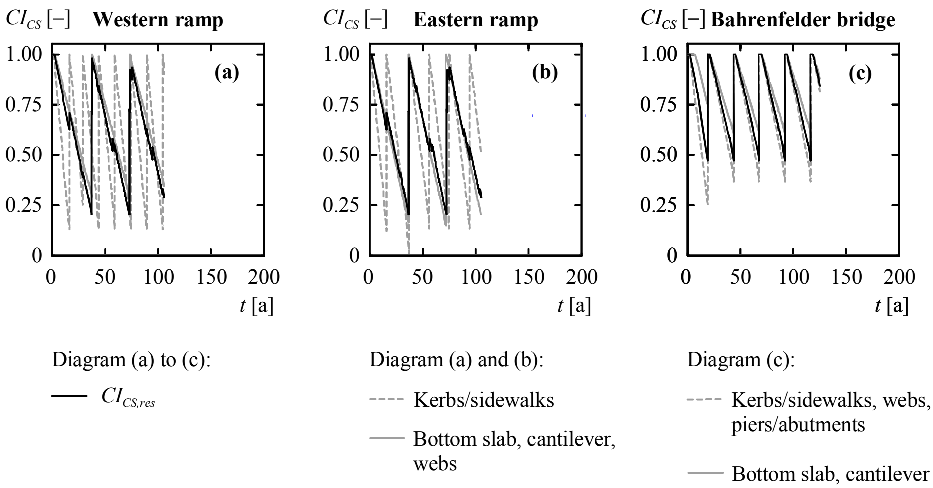



**Fig. 5.23:** Condition of the concrete surface: Time-dependent development for the *Köhlbrandbrücke* ((a) and (b)) and for the *Brücke Bahrenfelder Chaussee* (c).

In the case of the *Köhlbrandbrücke*, the following peculiarity should be mentioned. At first sight, the 50-percentile curve of the resulting concrete condition $CI_{CS,res}$ does not follow—in contrast to the expected outcome—an intermediate course with respect to the investigated sample with its $N$=100 different courses. However, a closer look reveals that the 50-percentile in fact follows an intermediate course. The seemingly off-centre position of the curve results from the maintenance application rules defined for this strategy: In the case that sidewalk deterioration is considerably faster than the deterioration of the girder concrete surface, sidewalk rehabilitation is applied before girder concrete repair. In this case, the small condition improvement delays the girder repair activities and thus leads to a double range of low points, of which the 50-percentile is in the middle of the first low point range (Fig. 5.24).

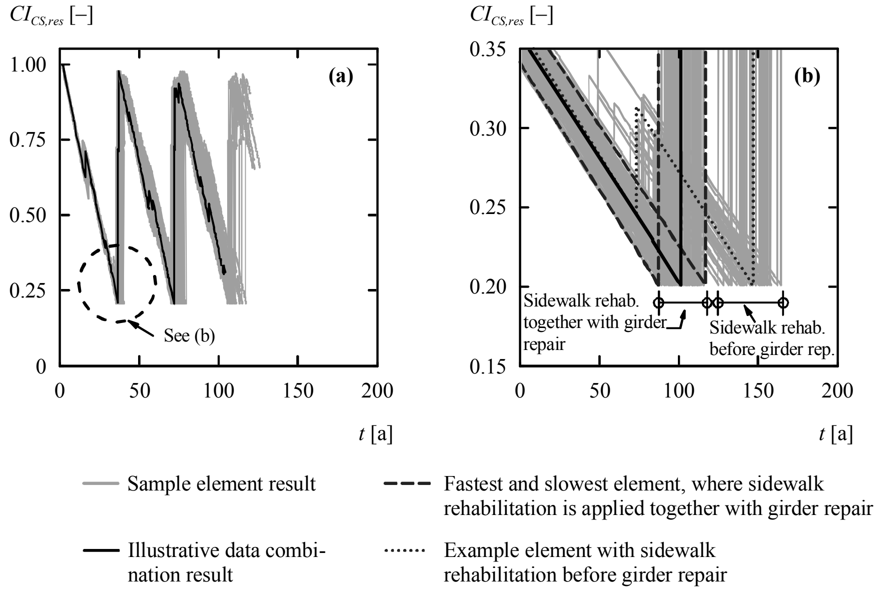



**Fig. 5.24:** Condition of the concrete surface: Detailed depiction of the development at the *Köhlbrandbrücke* during the time of the first major concrete repair works: (a) General depiction over the whole service life with the seemingly off-centre course of the illustrative data combination and (b) detailed depiction.

**Development of the Pavement Condition**

The pavement maintenance practice differs from bridge to bridge, both with respect to the applied maintenance activities and to the application thresholds. In the case of the *Köhlbrandbrücke*, the resulting pavement condition $CI_{A,res}$ according to Fig. 5.20 (d) combines the two distinct condition developments $CI_{A,i}$ along the eastern and western ramp, which are depicted in Fig. 5.25. In contrast, the pavement condition for the *Bahrenfelder* bridge is not derived from single spans but is expressed for the whole bridge, so that the resulting condition development $CI_{A,res}$ in Fig. 5.21 does not need further specification.

In the case of the *Köhlbrandbrücke*, the first major intervention is conducted in the form of bridge deck reconstruction works with complete pavement replacement at a performance level of $CI_{A,res}$=0.464. Later interventions beyond the known time horizon are assumed to be applied in the form of asphalt replacement along the whole bridge at an alternating threshold value of $CI_{A,res}$=0.375 and 0.464. The first value is chosen in the scope of the present investigation by considering the fact that no further major pavement maintenance has been applied until the year 2013. Therefore earliest intervention must happen later than 39 years after construction. The value $CI_{A,res}$=0.375 results from extrapolating the condition development to the year 2014. However, also a second threshold of 0.464 is introduced on an alternating base in order to account for the actually observed intervention practice.

In the case of the *Brücke Bahrenfelder Chaussee*, two pavement maintenance interventions were observed 18 and 36 years after bridge construction. At both occasions pavement resurfacing was applied along the complete bridge when an overall pavement condition of $CI_{A,res}$ =0.25 was reached (Fig. 5.21 (d)). However, in the scope of the present investigation it is assumed that after repeated resurfacing only a complete asphalt replacement can restore a sufficient pavement condition. Accordingly, the pavement maintenance plan assumes a cycle of repeated pavement resurfacing and replacement.

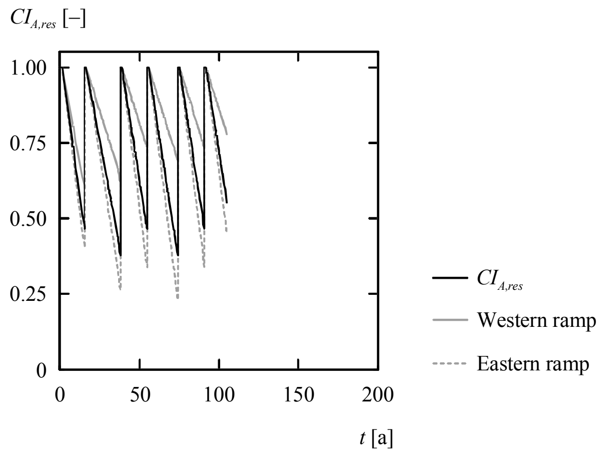



**Fig. 5.25:** Condition of the asphalt pavement: Time-dependent development for the Köhlbrandbrücke.

**Development of the Expansion Joint Condition**

For both bridges, the expansion joint condition development in terms of the resulting condition $CI_{Exp,res}$ shows a development with a cyclic alternation between the as new condition and lower threshold values. In both cases, two lower threshold values occur—the first being 0.25 for both bridges, the second being 0.53 for the *Köhlbrandbrücke* and 0.75 for the *Brücke Bahrenfelder Chaussee*. However, a characteristic difference occurs in the actual maintenance activities, which are applied at these thresholds. In the case of the Köhlbrandbrücke, expansion joint repair is applied first at the lower threshold value of $CI_{Exp,res}$=0.25 and replacement is only conducted after one more repair activity at the higher threshold level of 0.53. In the case of the *Bahrenfelder* bridge, the maintenance schedule applies a complete joint replacement first and only after that considers one consecutive repair activity at a higher threshold of $CI_{Exp,res}$=0.75.

The following Fig. 5.26 depicts the condition development for the *Köhlbrandbrücke*. Because the *Bahrenfelder* bridge is equipped with a single expansion joint, the resulting condition development $CI_{Exp,res}$ in Fig. 5.21 (e) does not need further specification.

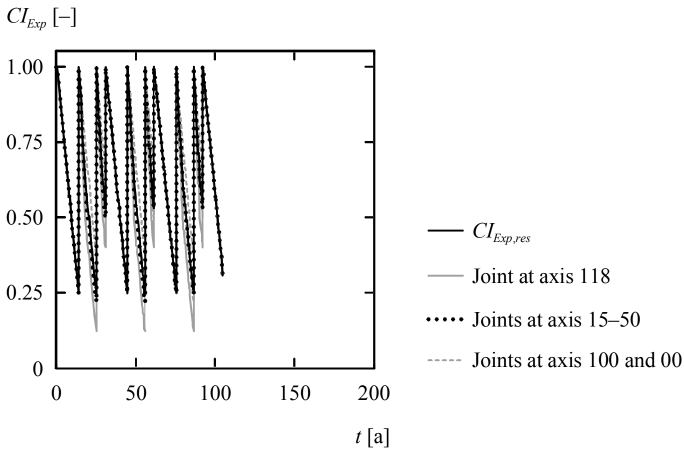



**Fig. 5.26:** Condition of the expansion joints: Time-dependent development for the Köhlbrandbrücke.

**Development of the Bearing Condition**

Obvious differences occur between the two bridges for the resulting bearing condition development ($CI_{bear,res}$). This is the case even if the underlying bearing maintenance strategy does not differ much from one bridge to the other, in so far as for both bridges bearing repair or replacement is applied when the worst deteriorated bearing reaches a condition of $CI_{Bear}$=0. However, clearly different courses for the $CI_{bear,res}$ development arise because of two reasons.

The first reason is that the SAMS-model of the *Köhlbrandbrücke* considers different bearing types—roller and pot-bearings—with considerably different deterioration rates and thus separate times of repair: In contrast to maintenance interventions at the bridge *Bahrenfelder Chaussee*, which affect all bearings of the bridge at the same time, maintenance is selectively applied to the eastern and western ramp in the case of the Köhlbrandbrücke. As a result, the overall bearing condition development of the *Köhlbrandbrücke* for this strictly reactive strategy is not of the type '1–0–1–...' but is rather characterised by smaller improvements, as can be seen from Fig. 5.27.

The second specific difference between the two condition developments occurs because of the different deterioration data for the bridge bearings, with a very long initiation period for the Brücke Bahrenfelder Chaussee and none in the case of the Köhlbrandbrücke.

The following Fig. 5.27 depicts the condition development for the *Köhlbrandbrücke*. Because the bearings of *Bahrenfelder* bridge are represented by a single SAMS-element so that

$CI_{Bear}$ is equal to $CI_{Bear,res}$ and consequently the resulting condition development in Fig. 5.21 (f) does not need further specification.

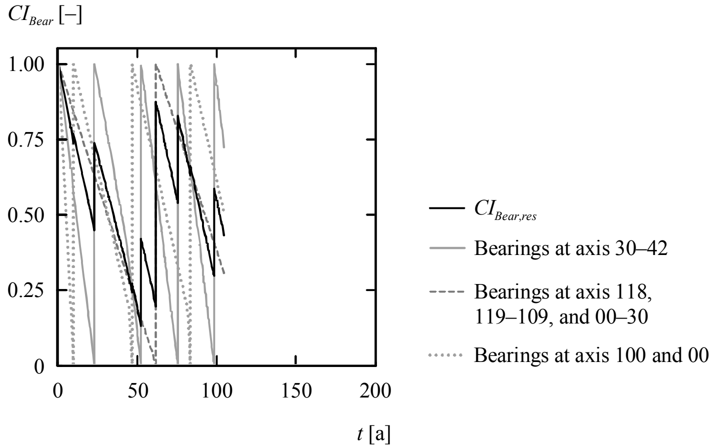



**Fig. 5.27:** Condition of the bearings: Time-dependent development for the Köhlbrandbrücke.

**Development of the Drainage System Condition**

For both bridges, the entire drainage system is represented by a single SAMS-element so that further specification beyond the condition development courses in Fig. 5.20 (f) and Fig. 5.21 (f) is not needed.

### 5.7.3 Strategy $S_1$ – Reactive Repair Strategy

The condition results for the reactive repair strategy show a characteristic pattern in the form of an oscillation between very well condition indices on the one side and critical condition indices on the other side (Fig. 5.28, Fig. 5.29). As a result, this strategy leads to the worst average condition over the service life compared to all the other investigated strategies. This simply follows from the fact that maintenance interventions are applied as late as possible.

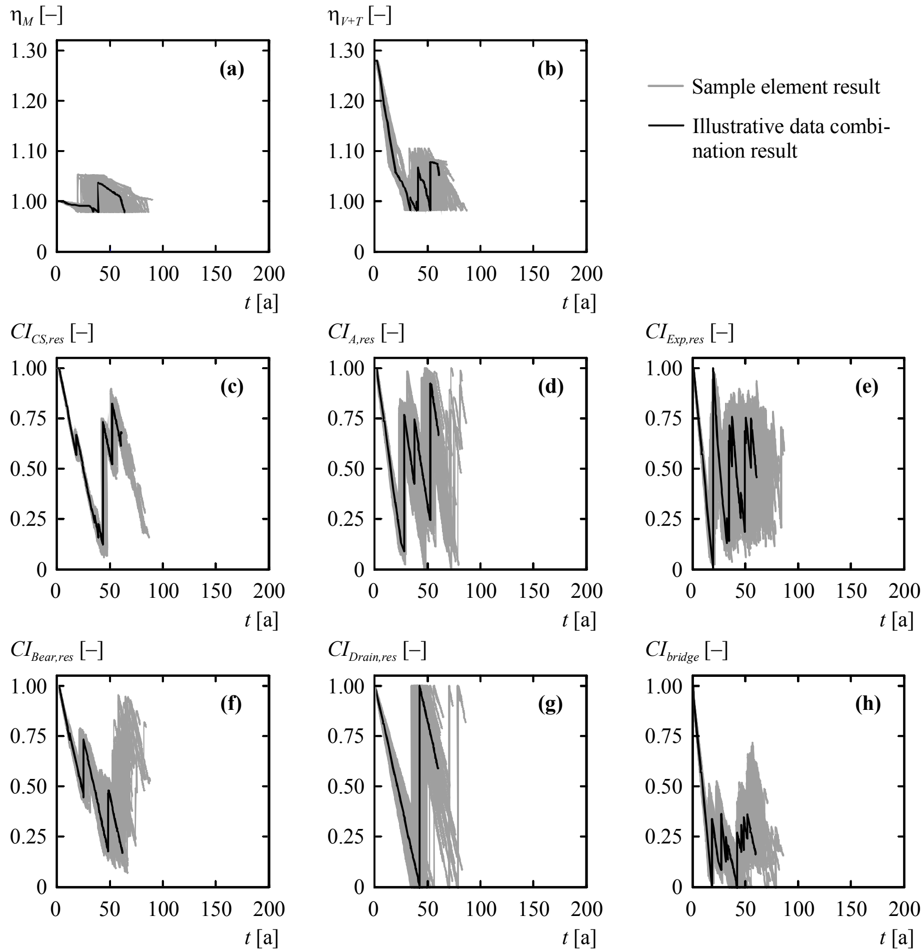



**Fig. 5.28:** Strategy $S_1$ (Köhlbrandbrücke): Simulation results for each one of the $N$=100 sample elements (grey lines) and the illustrative data combination (black line).

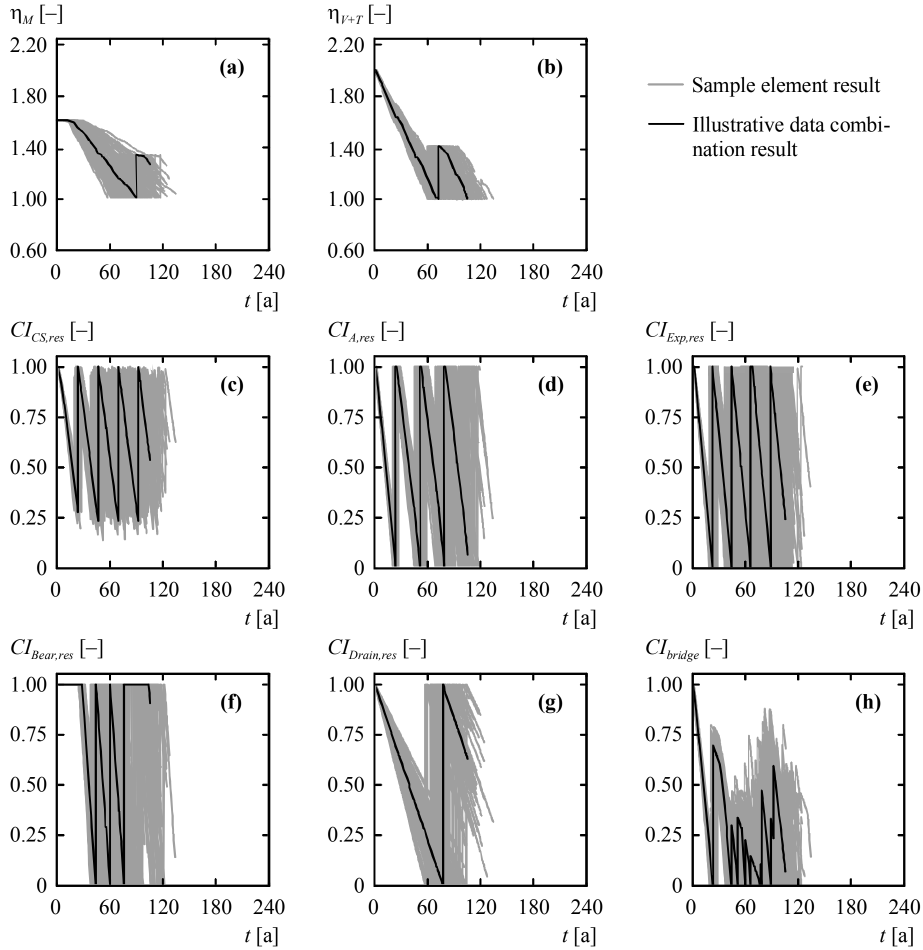



**Fig. 5.29:** Strategy $S_1$ (Bahrenfelder Chaussee): Simulation results for each one of the $N$=100 sample elements (grey lines) and the illustrative data combination (black line).

The example of the *Brücke Bahrenfelder Chaussee* provides a clear demonstration for the characteristic condition development of a reactive repair strategy. Each one of the condition indices $CI_{A,res}$, $CI_{Exp,res}$, $CI_{bear,res}$, and $CI_{Drain,res}$ perfectly follows an alternating course with bottom and peak values of 0 and 1, respectively. Only the concrete surface condition shows a different development in such a way that maintenance intervention occurs before a resulting condition $CI_{CS,res}$ of 0 occurs. The explanation for this follows from the detailed concrete condition development of the webs, bottom slab, the cantilevers, the sidewalks, and the substructure in Fig. 5.30 (c): In the scope of the present investigation the resulting concrete surface condition is calculated as the weighted average of five different condition values—those of the webs, the bottom slab, the deck cantilevers, the sidewalks, and the substructure. According to the maintenance application rules for strategy $S_1$, maintenance intervention is applied when any of these indicators reaches zero. However, because these concrete elements show different deterioration rates according to the inspection data, the average value $CI_{CS,res}$ is better than the worst element's condition. Therefore, maintenance intervention occurs at a resulting condition above zero. Yet, because the strategy definition provides complete concrete rehabilitation works—girder, sidewalks, and abutments—each maintenance interventions leads to the as new value of $CI_{CS,res}$=1.0. This effect does not occur with the other resulting condition indicators, for example with respect to the pavement or the bearings, because they are represented by one single SAMS-Element and do not condense different developments.

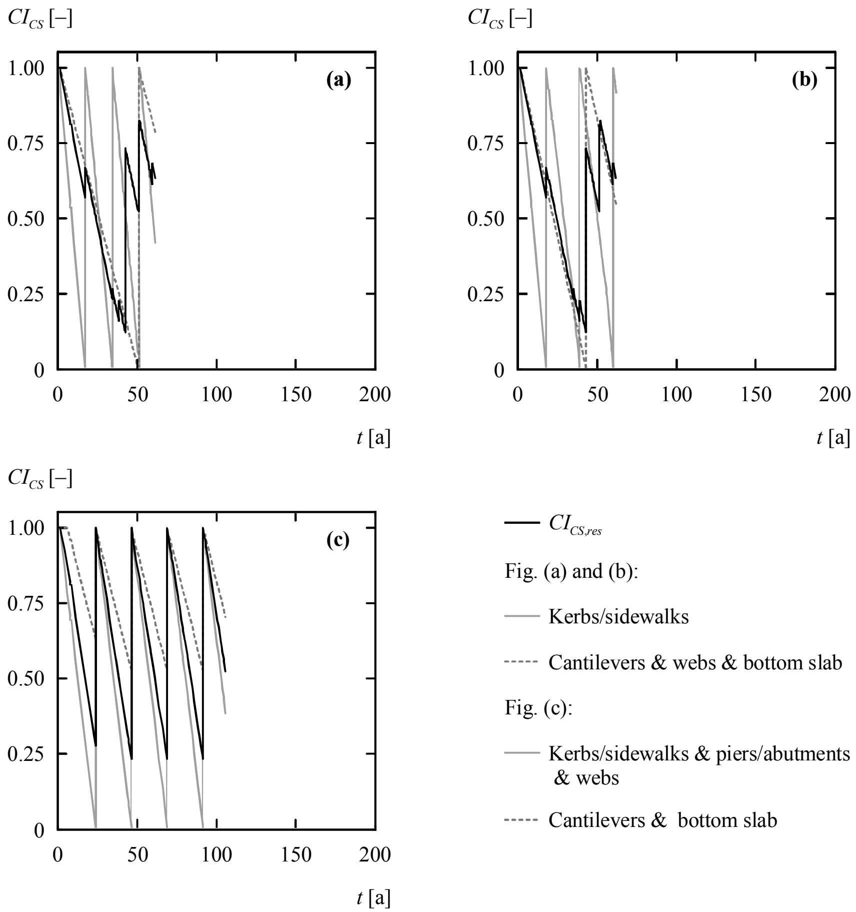



**Fig. 5.30:** Condition of the concrete surface: Time-dependent development for the Köhlbrandbrücke (eastern ramp (a), eastern ramp (b)) and for the Brücke Bahrenfelder Chaussee (c).

The above described effects are generally observed for the *Köhlbrandbrücke*—however, a slightly different picture appears for the drainage system condition development. This is because—unlike the *Bahrenfelder Chaussee* bridge—each one of the resulting condition indices aggregates at least two separate developments. For example, the resulting pavement condition condenses the condition developments at the eastern and western ramp. The resulting expansion joint as well as the resulting bearing condition development is derived from even more different courses. The resulting expansion joint condition is derived from the five individual condition developments of the SAMS-elements '118', '117', '101', '00', and '57.9' and the axis range 15–50. Similarly, the bearing condition development refers to different bearing types—roller and pot-bearings—with considerably different deterioration rates, which are represented by 6 different condition developments. Consequently, the resulting condition developments does not show a constant lower value of *CI*=0. Moreover, maintenance interventions usually do not restore the resulting condition to an as-new value of 1.0 because the maintenance activities are selectively applied to individual ramps. In this case, the resulting condition cannot completely be restored to 1.0 because the unmaintained elements with their sub-optimal condition also influence the resulting value.

### 5.7.4 Strategy $S_2$ – Proactive Repair Strategy

In the case of the *proactive repair strategy* $S_2$, both bridges show a comparable condition development with an oscillating course, which—in its general shape—is similar to the above described reactive strategy $S_1$. However, following from the maintenance application rules, the characteristic condition bandwidth is not from 'as new' to 'poor', as it is in the case of the reactive strategy, but the lower condition bound is in the range of 'fair' condition values. As a result, as can be seen from the below Fig. 5.31 and Fig. 5.32, this strategy leads to a better average condition compared to the reactive strategy $S_1$. However, this is bought by more frequent maintenance application.

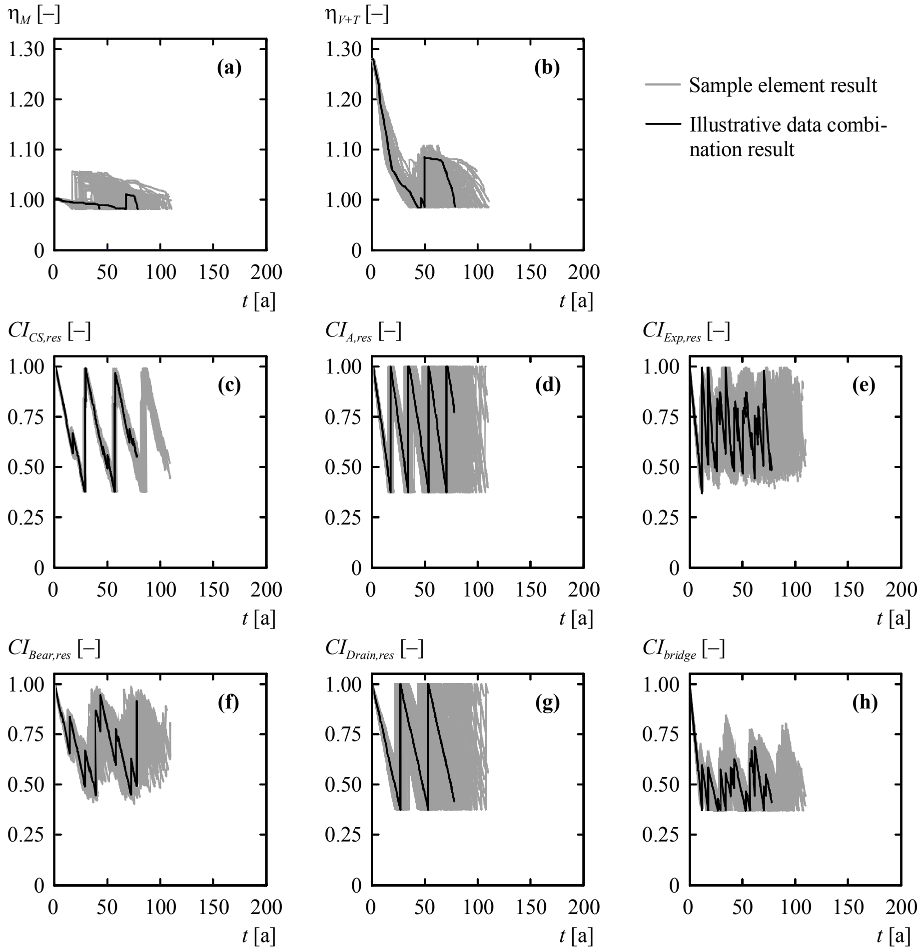



**Fig. 5.31:** Strategy $S_2$ (Köhlbrandbrücke): Simulation results for each one of the $N$=100 sample elements (grey lines) and the illustrative data combination (black line).

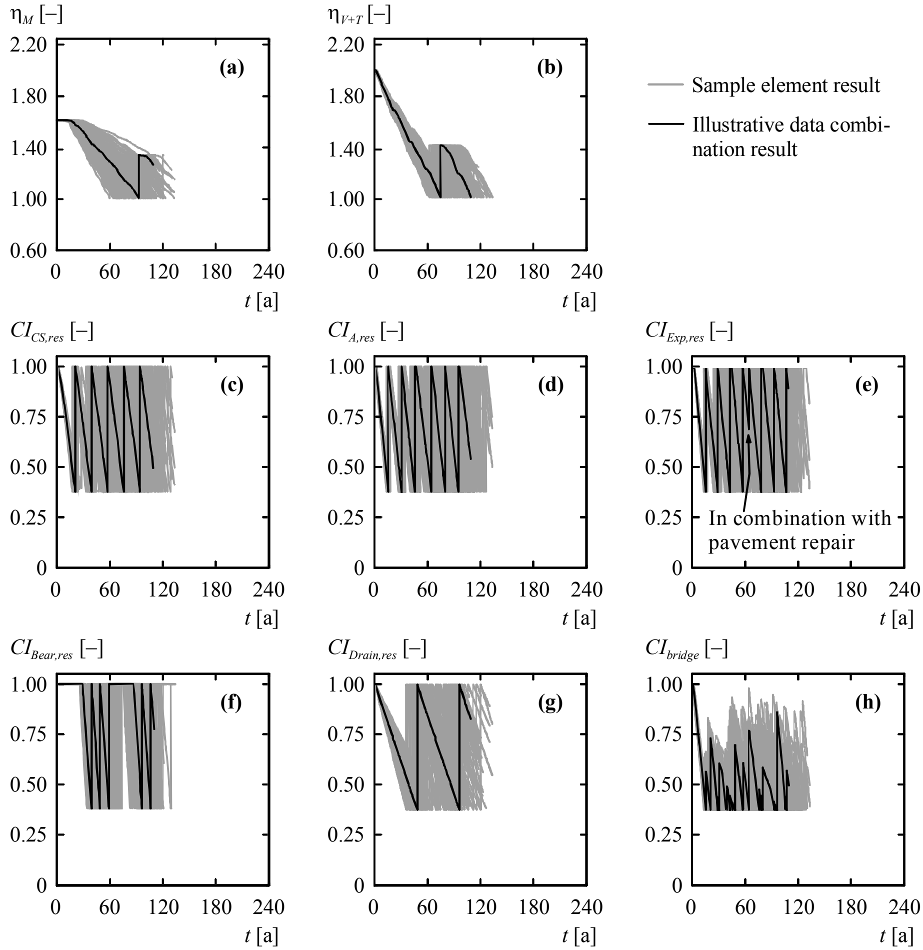



**Fig. 5.32:** Strategy $S_2$ (Bahrenfelder Chaussee): Simulation results for each one of the $N$=100 sample elements (grey lines) and the illustrative data combination (black line).

It is found for the *Brücke Bahrenfelder Chaussee* that the resulting condition indicator development of the bridge equipment perfectly follows a course that oscillates between 'as new' and 'fair' condition with clear and constant upper and lower bounds of 1.0 and 0.375 respectively (Fig. 5.32). As it is the case with the above described strategies, these rather idealised courses follow from the fact that the SAMS model for the *Bahrenfelder* bridge uses only a single element for each part of the bridge equipment so that—self-evidently—the resulting condition is equal to the condition value at the SAMS element that represents the bridge equipment in question. Exceptions to the lower condition bounds only occur for the expansion joint condition, where maintenance is proactively applied together with pavement intervention works, as it happens for example in the case of the illustrative data combination about 70 years after bridge construction (Fig. 5.32 (e)).

For the *Köhlbrandbrücke*, the resulting condition of the concrete surface and the pavement follows a rather straight course which is also similar to that of the *Brücke Bahrenfelder Chaussee*. This is because proactive pavement maintenance and concrete repair works are globally applied to the whole bridge and not, as for the strategy $S_1$, locally to the more seriously deteriorated ramp. An example for this is depicted in the below Fig. 5.33. As it can be seen in Fig. 5.33 (a) and (b), proactive concrete girder repair is applied to both ramps in combination with sidewalk rehabilitation when a resulting concrete condition of 0.375 is reached. Similarly, pavement maintenance activities are applied to both ramps at a threshold value of $CI_{A,res}$ = 0.375. These harmonised interventions in effect decouple the maintenance regime in large parts from local influences. As a result, the major proactive maintenance interventions restore the pavement and concrete surface condition back to as-new condition.

However, sidewalk concrete rehabilitation still occurs on a local basis because of the very quick sidewalk deterioration progress, which necessitates reactive sidewalk repair in addition to the proactive interventions (Fig. 5.33 (a) and (b)). This also causes exceptions to the harmonised concrete repair works: It can be seen in Fig. 5.33 (b) that sidewalk rehabilitation to the eastern ramp is not applied together with the other concrete repair works around the year 60. This is because, according to the strategy maintenance application rules, sidewalk rehabilitation is only applied in combination with girder repair when the last sidewalk repair works lay back more than 10 years.

The expansion joint and bearing condition developments of the *Köhlbrandbrücke* still show a rather turbulent pattern. This is because for these bridge equipment parts intervention activities are applied exclusively to the most deteriorated elements at which the threshold violation actually occurs. Naturally, the stricter condition threshold leads to more frequent interventions compared to the reactive strategy $S_1$. Also, the proactive expansion joint maintenance together with pavement interventions—applied to those joints whose latest maintenance lays back more than 10 years—additionally increases the number of maintenance interventions.

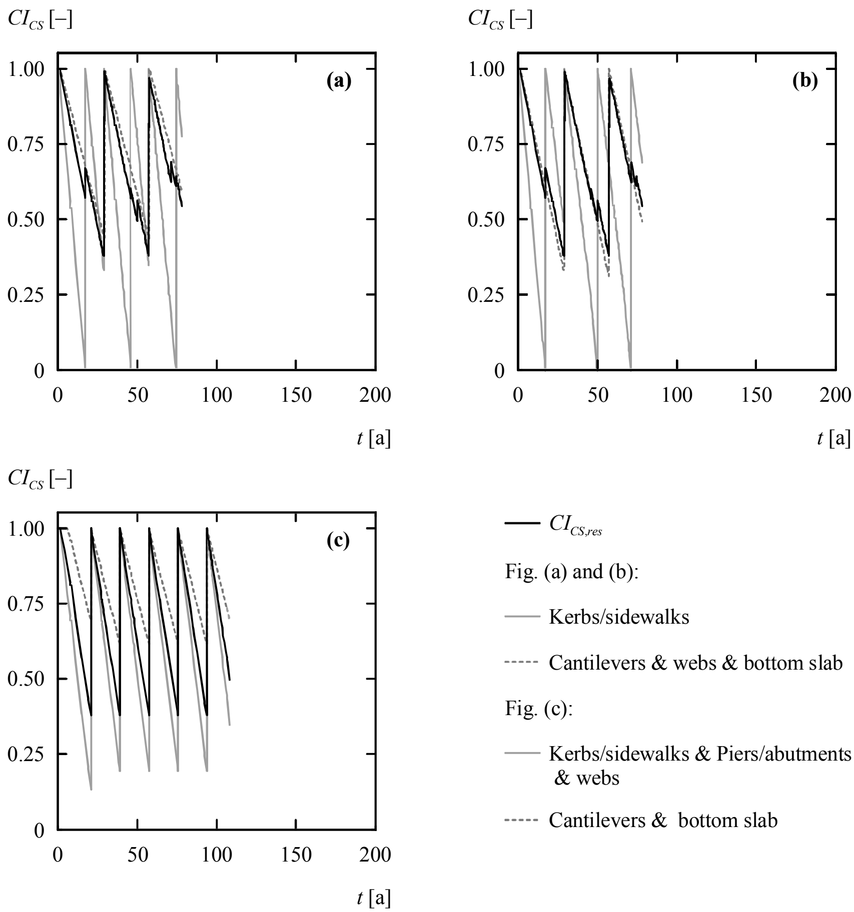



**Fig. 5.33:** Condition of the concrete Surface: Time-dependent development for the Köhlbrandbrücke (eastern ramp (a), eastern ramp (b)) and for the Brücke Bahrenfelder Chaussee (c).

### 5.7.5 Strategy $S_3$ – Preventive Repair

For both the *Köhlbrandbrücke* (Fig. 5.34) and the *Brücke Bahrenfelder Chaussee* (Fig. 5.35) the condition development widely resembles the reactive condition development of the strategy $S_1$. This is rather unsurprising because the preventive repair strategy $S_3$ is simply defined as the reactive repair strategy $S_1$ with additional preventive hydrophobic coating and intensified routine maintenance.

Yet, the decisive difference is that the deterioration curves for elements which are directly affected from the preventive activities are stretched over time. By far the highest delaying effect applies to the concrete surface condition development. In contrast, the condition development of the bearings and the drainage system is mostly unchanged.

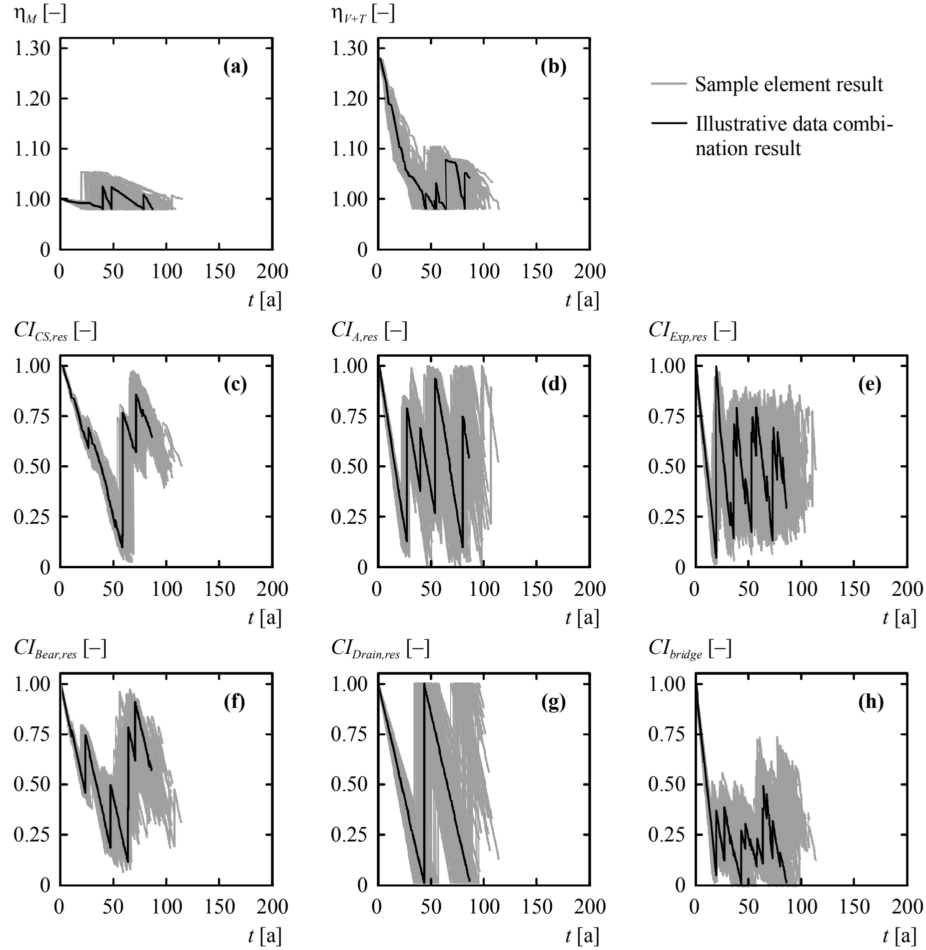



**Fig. 5.34:** Strategy $S_3$ (Köhlbrandbrücke): Simulation results for each one of the $N$=100 sample elements (grey lines) and the illustrative data combination (black line).

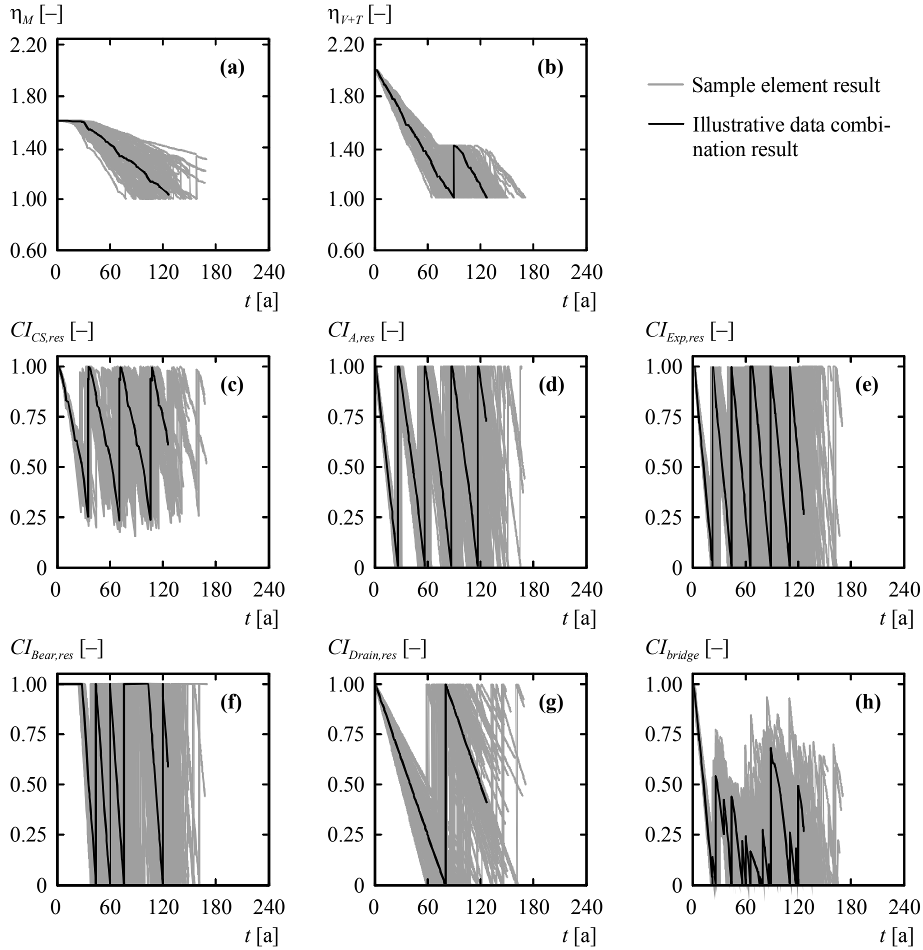



**Fig. 5.35:** Strategy $S_3$ (Bahrenfelder Chaussee): Simulation results for each one of the $N$=100 sample elements (grey lines) and the illustrative data combination (black line).

### 5.7.6 Strategy $S_4$ – Preventive-Proactive Strategy

According to its definition in sub-section 5.6.2, the preventive-proactive repair strategy $S_4$ combines the intervention rules of the proactive repair strategy $S_2$ with additional preventive hydrophobic coating and intensified routine maintenance. Accordingly, the resulting condition development for both the *Köhlbrandbrücke* (Fig. 5.36) and the *Brücke Bahrenfelder Chaussee* (Fig. 5.37) resembles the proactive condition development with stretched deterioration curves for elements that are directly affected from the preventive activities.

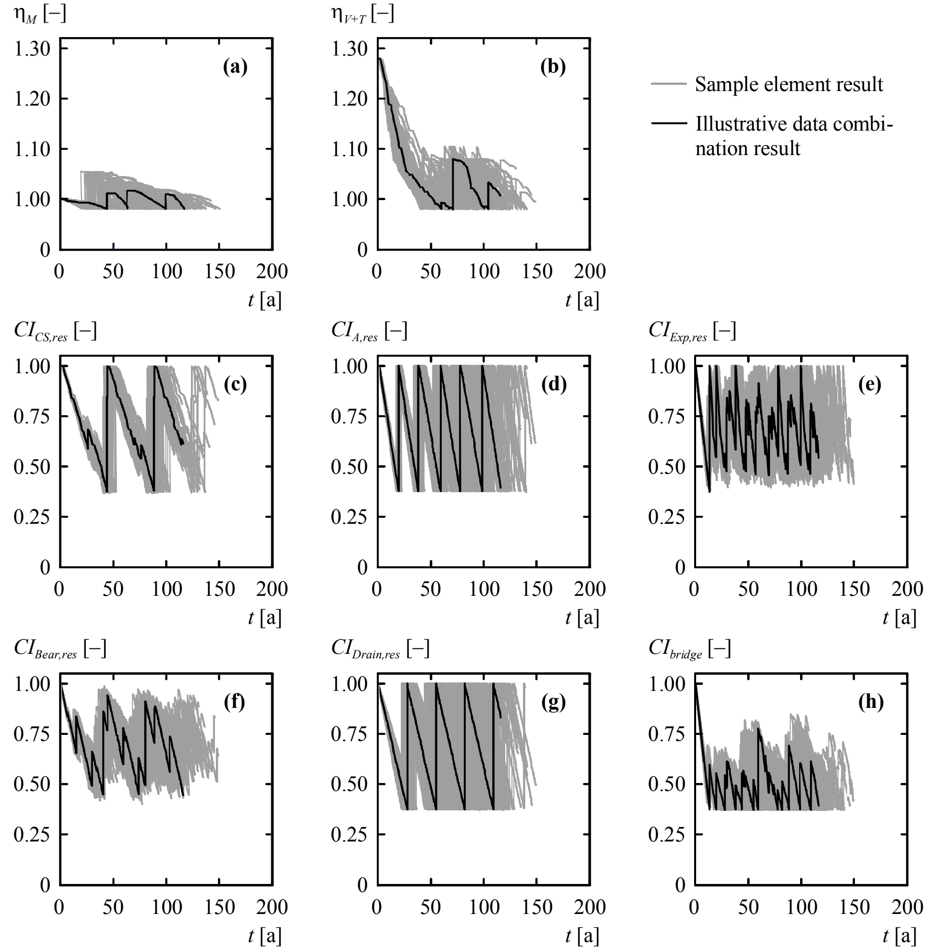



**Fig. 5.36:** Strategy $S_4$ (Köhlbrandbrücke): Simulation results for each one of the $N$=100 sample elements (grey lines) and the illustrative data combination (black line).

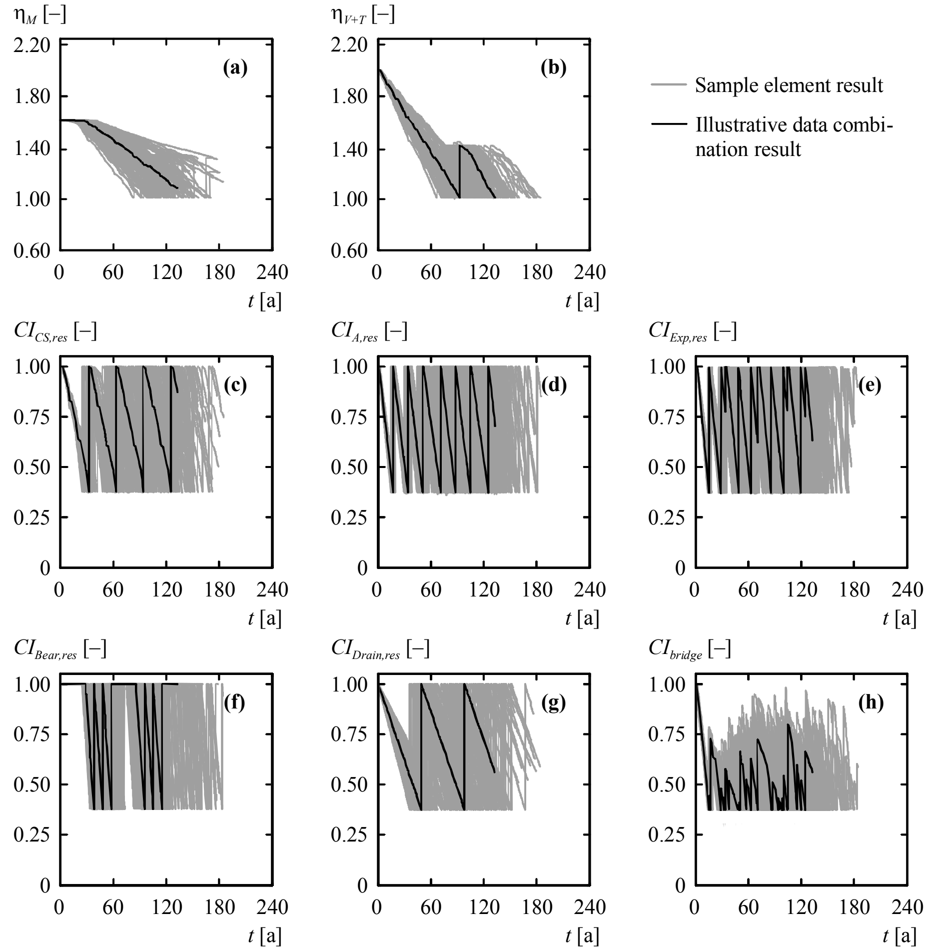



**Fig. 5.37:** Strategy $S_4$ (Bahrenfelder Chaussee): Simulation results for each one of the $N$=100 sample elements (grey lines) and the illustrative data combination (black line).

### 5.7.7 Strategy $S_{5.1}$ – Traffic Performance

According to its definition in sub-section 5.6.2, the strategy $S_{5.1}$ combines the intervention rules of the repair strategy $S_1$ with highly proactive expansion joint and pavement maintenance and with intensified routine minor maintenance in order to reduce traffic flow restrictions.

The below Fig. 5.38 and Fig. 5.39 both show that this strategy successfully provides a continuously high-level condition of the pavement and the expansion joints. The other elements, such as the bearings or the concrete surface, are merely reactively maintained and therefore only the intensified application of routine maintenance leads to marginally reduced deterioration rates of these elements. Their condition development corresponds almost perfectly to that of the reactive strategy $S_1$ in the above Fig. 5.28 and Fig. 5.29.

However, the development of $\eta_{V+T}$ appears different from the reactive strategy $S_1$. This is because expansion joint maintenance positively affects the corrosion as well as concrete strength degradation processes around the expansion joint. The bending capacity $\eta_M$ is not affected because of the fact that expansion joints are located at the free ends of girders—thus, neither bending in mid-span or over the piers is influenced by steel and concrete corrosion around the expansion joints.

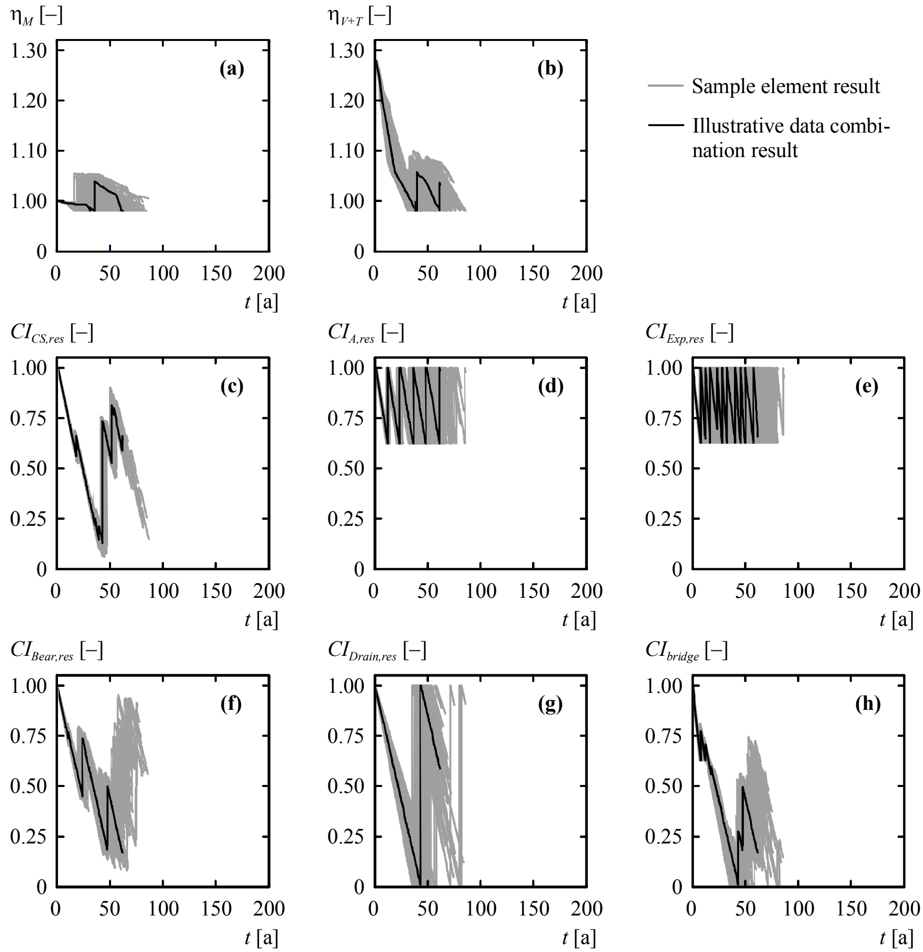



**Fig. 5.38:** Strategy $S_{5.1}$ (Köhlbrandbrücke): Simulation results for each one of the $N$=100 sample elements (grey lines) and the illustrative data combination (black line).

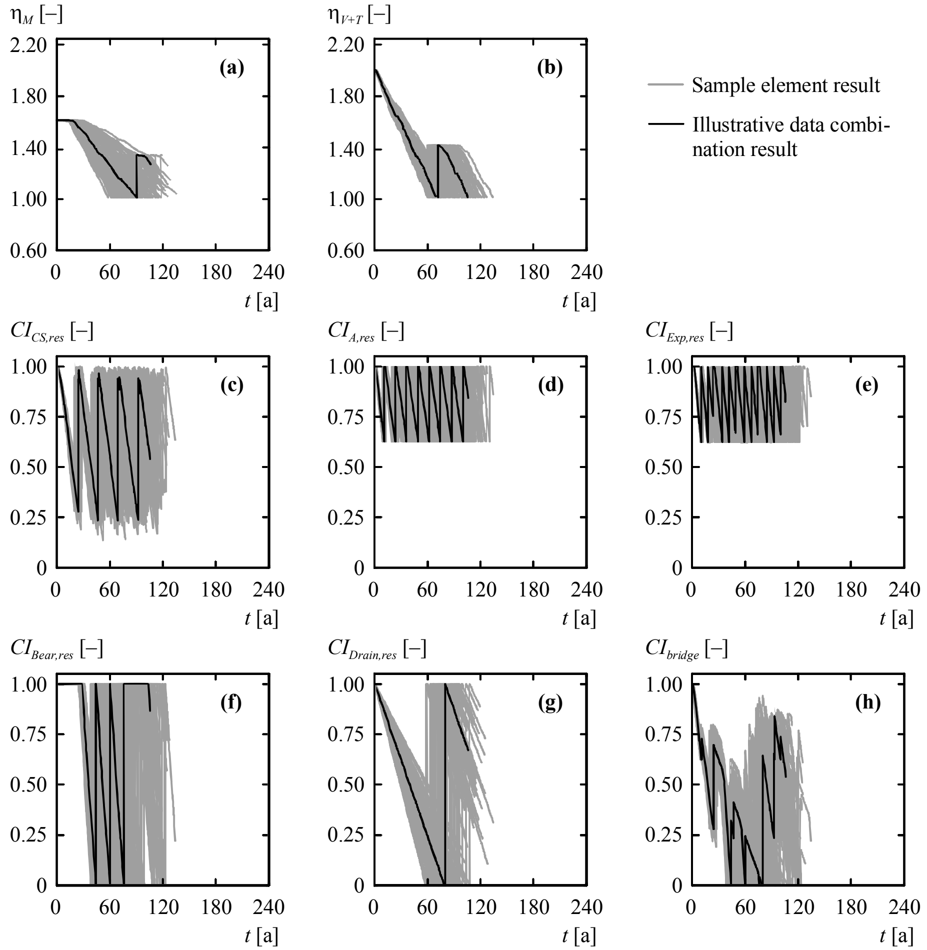



**Fig. 5.39:** Strategy $S_{5.1}$ (Bahrenfelder Chaussee): Simulation results for each one of the $N$=100 sample elements (grey lines) and the illustrative data combination (black line).

### 5.7.8 Strategy $S_{5.2}$ – Visual Appearance

As it is intended by this strategy, the condition of the visible concrete surface, which determines the overall visual appearance, shows to be continuously good. Compared to all the other investigated strategies, the resulting condition development of the concrete surface clearly stands out with its continuously high levels.

The asphalt condition development exactly corresponds to the results for the preventive proactive-preventive strategy $S_4$. As opposed to this, the expansion joint condition development just corresponds in terms of the lower condition bound with the strategy $S_4$; whereas, characteristic differences between both strategies arise because of the fact that the strategy $S_{5.2}$ does not apply proactive expansion joint maintenance together with asphalt maintenance works. The other condition courses show comparative results to the previously described strategies. On the one hand, both the bearing and the drainage system condition development, almost perfectly corresponds to the developments according to the preventive repair strategy $S_3$; this is simply because condition threshold values correspond for both strategies and also both apply intensified routine maintenance.

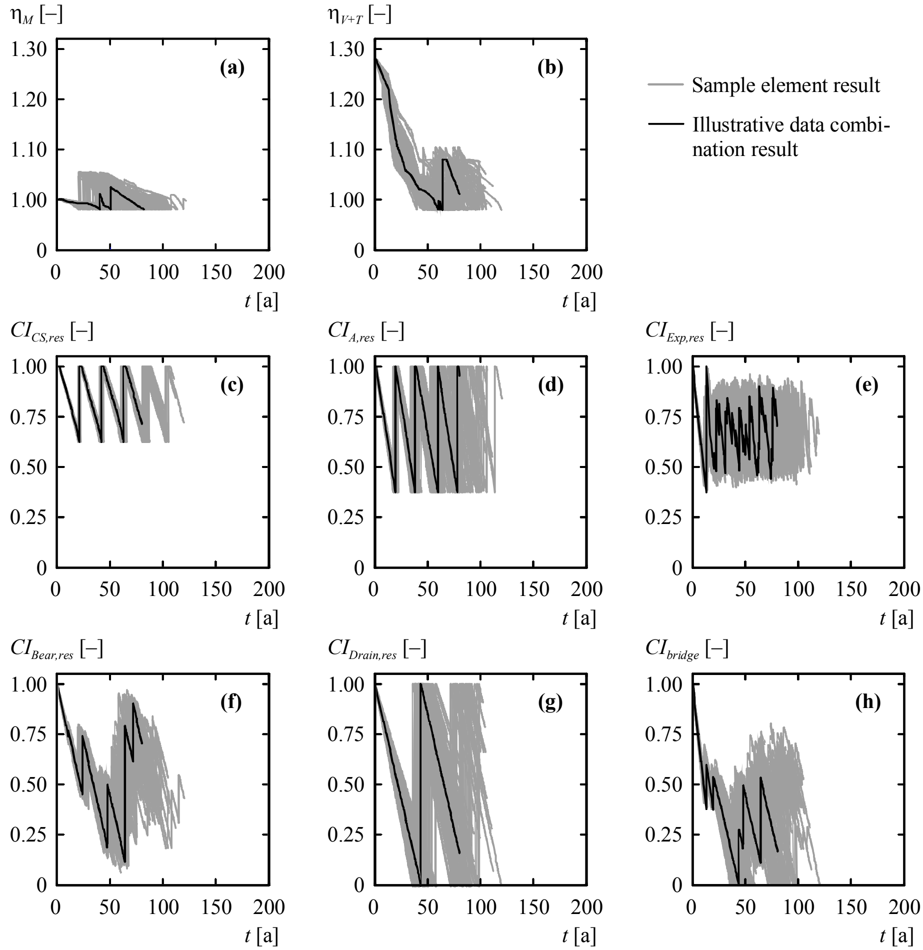



**Fig. 5.40:** Strategy $S_{5.2}$ (Köhlbrandbrücke): Simulation results for each one of the $N$=100 sample elements (grey lines) and the illustrative data combination (black line).

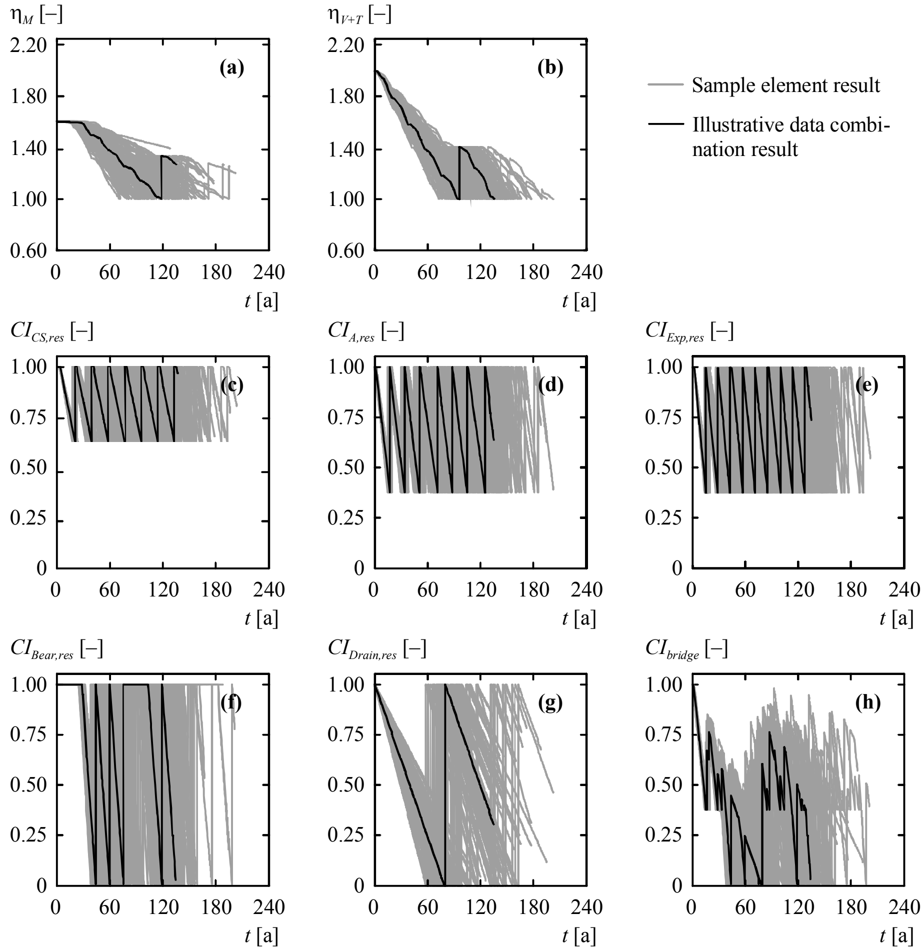



**Fig. 5.41:** Strategy $S_{5.2}$ (Bahrenfelder Chaussee): Simulation results for each one of the $N$=100 sample elements (grey lines) and the illustrative data combination (black line)

### 5.7.9 Strategy $S_{5.3}$ – Structural Condition

In a quite distinctive way, the effects of the strong focus on a sound structural condition become apparent for the *Köhlbrandbrücke* in Fig. 5.42 (a) and (b) and for the *Brücke Bahrenfelder Chaussee* in Fig. 5.43 (a) and (b): There, the proactive conservation works, which are applied together with concrete rehabilitations, occur in the form of kinks in the deterioration curves of $\eta_M$ and $\eta_{V+T}$. Also, two striking proactive interventions, which include proactive tendon conservation and concrete repair works can be observed for the Köhlbrandbrücke around 28 years and 132 years after construction in Fig. 5.42 (a) and (c).

Also, the high demands on the condition of the concrete surface and the bridge equipment are successfully implemented, as it can be seen from the mainly better than fair condition levels (Fig. 5.42 and Fig. 5.43).

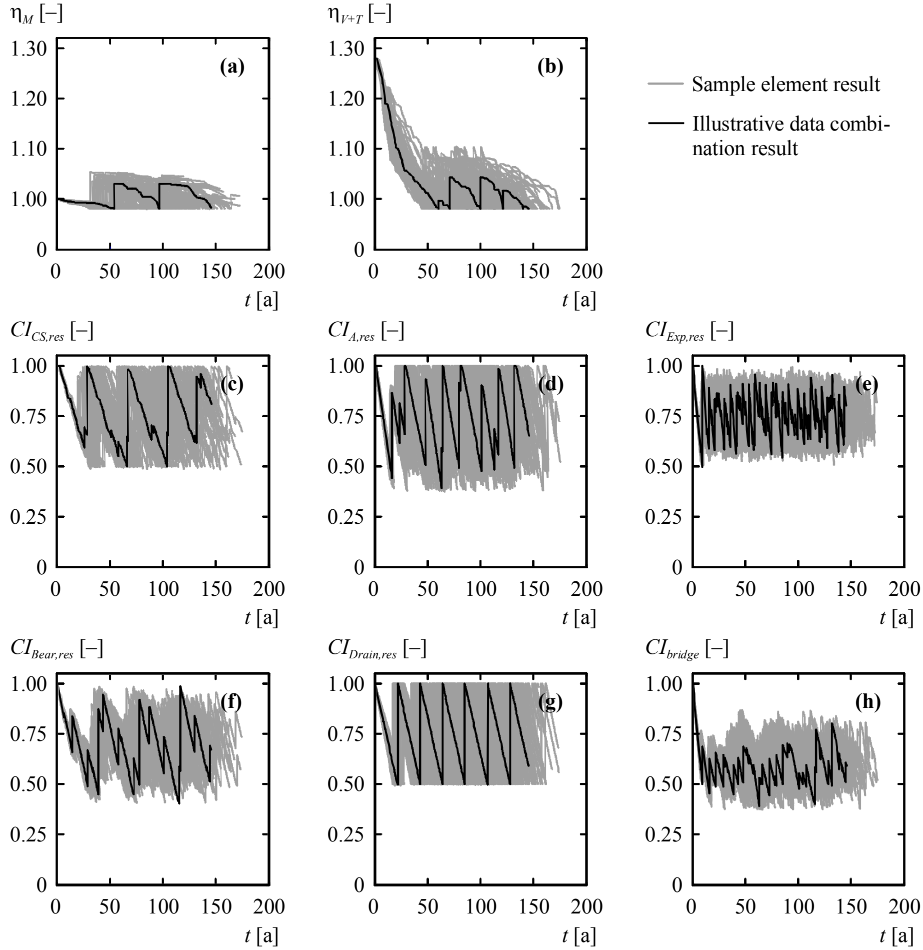



**Fig. 5.42:** Strategy $S_{5.3}$ (Köhlbrandbrücke): Simulation results for each one of the $N$=100 sample elements (grey lines) and the illustrative data combination (black line).

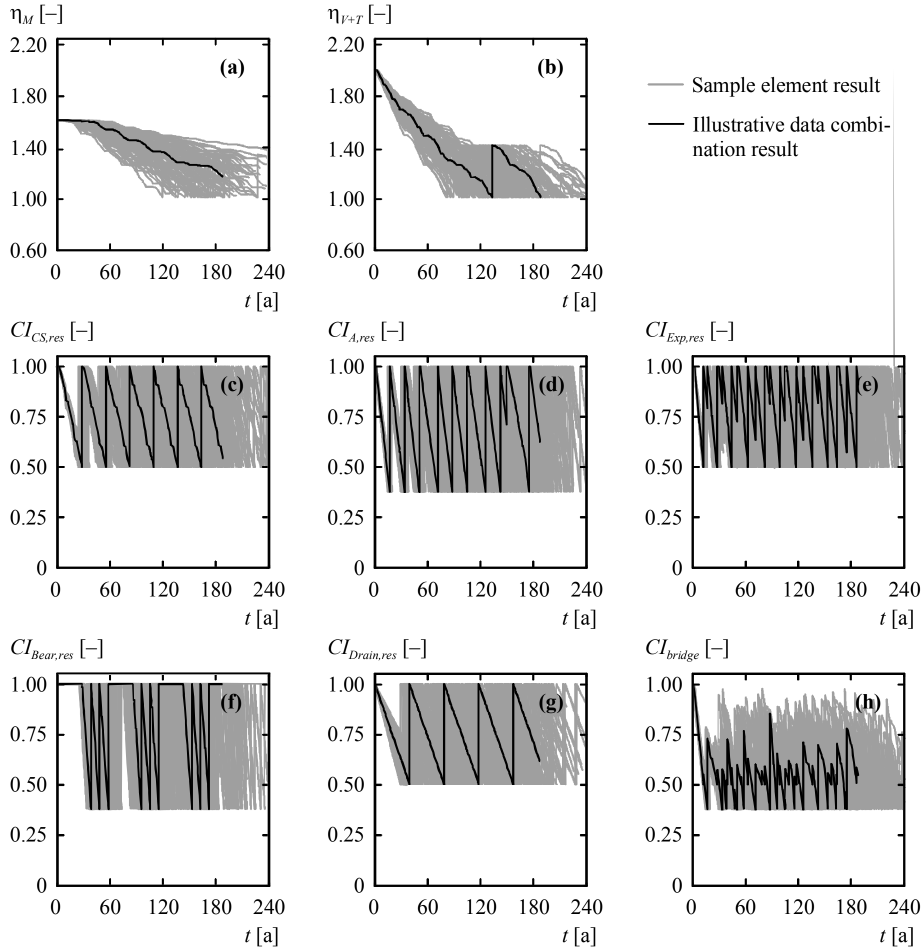



**Fig. 5.43:** Strategy $S_{5.3}$ (Bahrenfelder Chaussee): Simulation results for each one of the $N$=100 sample elements (grey lines) and the illustrative data combination (black line).

### 5.7.10 Resulting Performance Indictor Values

Strategy assessment is achieved with respect to four resulting indicators, which express the functional, economic and societal as well as cultural implications of the investigated strategies: $T_{SL}$, $C_{0,owner}$, $A_{av}$, and $RT_{ua}$. The stochastic simulation with its $N$=100 combinations of input parameters does not provide precise resulting indicator values, but rather gives value distributions for each performance indicator (Fig. 5.44, Fig. 5.45). Following basic concepts of stochastic strategy assessment (e.g. Law 2009), in the scope of the present investigation, the value distributions are represented by the numerical parameters *sample mean value* $\mu_s$ and the *standard deviation* $\sigma$. Also, the 95% confidence interval for the expected mean value, which describes the interval around $\mu_s$ in which the true mean value of the distribution will be located with a 95% probability, is derived from the simulation results.

**Table 5-14:** Performance Indicator Distributions (Köhlbrandbrücke)

| | $T_{SL}$ [a] | | $C_{0,owner}$ [$10^6$ €] | | $RT_{UA}$ [–] | | $A_{av}$ [–] | |
|---|---|---|---|---|---|---|---|---|
| **Strat.** | $\mu$ [a] | $\sigma$ | $\mu$ [a] | $\sigma$ | $\mu$ [a] | $\sigma$ | $\mu$ [a] | $\sigma$ |
| $S_0$ | 99 ± 3 | 14 | 15.7 ± .3 | 1.5 | .70 ± .005 | .03 | .80 ± .001 | .006 |
| $S_1$ | 61 ± 2 | 9 | 18.3 ± .6 | 2.9 | .79 ± .006 | .03 | .74 ± .002 | .009 |
| $S_2$ | 82 ± 3 | 16 | 17.6 ± .6 | 3.1 | .61 ± .004 | .02 | .83 ± .001 | .006 |
| $S_3$ | 85 ± 3 | 14 | 12.7 ± .5 | 2.3 | .71 ± .008 | .04 | .74 ± .002 | .009 |
| $S_4$ | 108 ± 4 | 18 | 13.0 ± .3 | 1.4 | .56 ± .005 | .03 | .82 ± .001 | .005 |
| $S_{5.1}$ | 63 ± 2 | 8 | 20.6 ± .5 | 2.7 | .79 ± .006 | .03 | .89 ± .001 | .006 |
| $S_{5.2}$ | 84 ± 3 | 14 | 13.1 ± .4 | 2.1 | .24 ± .003 | .02 | .84 ± .001 | .006 |
| $S_{5.3}$ | 131 ± 4 | 20 | 15.7 ± .3 | 1.3 | .44 ± .012 | .06 | .85 ± .001 | .004 |

[a] The mean value is expressed in terms of its 95% confidence interval.

**Table 5-15:** Performance Indicator Distributions (Brücke Bahrenfelder Chaussee)

| | $T_{SL}$ [a] | | $C_{0,owner}$ [$10^3$ €] | | $RT_{UA}$ [–] | | $A_{av}$ [–] | |
|---|---|---|---|---|---|---|---|---|
| **Strat.** | $\mu$ [a] | $\sigma$ | $\mu$ [a] | $\sigma$ | $\mu$ [a] | $\sigma$ | $\mu$ [a] | $\sigma$ |
| $S_0$ | 124 ± 3 | 13 | 594 ± 9 | 44 | .457 ± .003 | .02 | .845 ± .001 | .007 |
| $S_1$ | 105 ± 2 | 9 | 529 ± 11 | 54 | .646 ± .005 | .03 | .772 ± .003 | .013 |
| $S_2$ | 109 ± 2 | 10 | 626 ± 12 | 59 | .588 ± .004 | .02 | .881 ± .001 | .008 |
| $S_3$ | 128 ± 3 | 16 | 375 ± 8 | 42 | .545 ± .008 | .04 | .766 ± .003 | .015 |
| $S_4$ | 134 ± 4 | 19 | 424 ± 8 | 42 | .510 ± .006 | .03 | .879 ± .001 | .007 |
| $S_{5.1}$ | 105 ± 2 | 9 | 610 ± 11 | 55 | .640 ± .005 | .03 | .928 ± .001 | .003 |
| $S_{5.2}$ | 138 ± 4 | 19 | 363 ± 6 | 30 | .308 ± .003 | .02 | .870 ± .001 | .007 |
| $S_{5.3}$ | 187 ± 7 | 33 | 539 ± 9 | 48 | .514 ± .004 | .02 | .888 ± .001 | .003 |

[a] The mean value is expressed in terms of its 95% confidence interval.

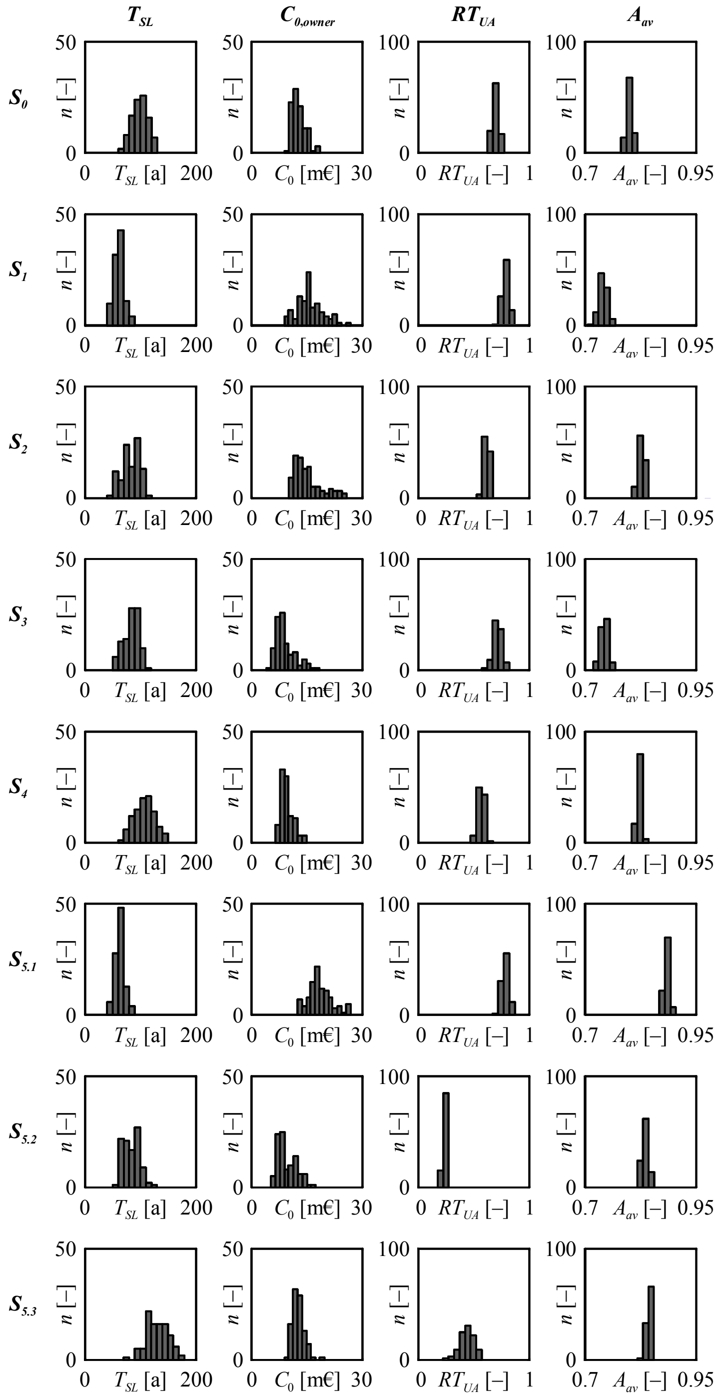



**Fig. 5.44:** Resulting distribution functions of the performance indicators for a sample size $N$=100 (Köhlbrandbrücke)

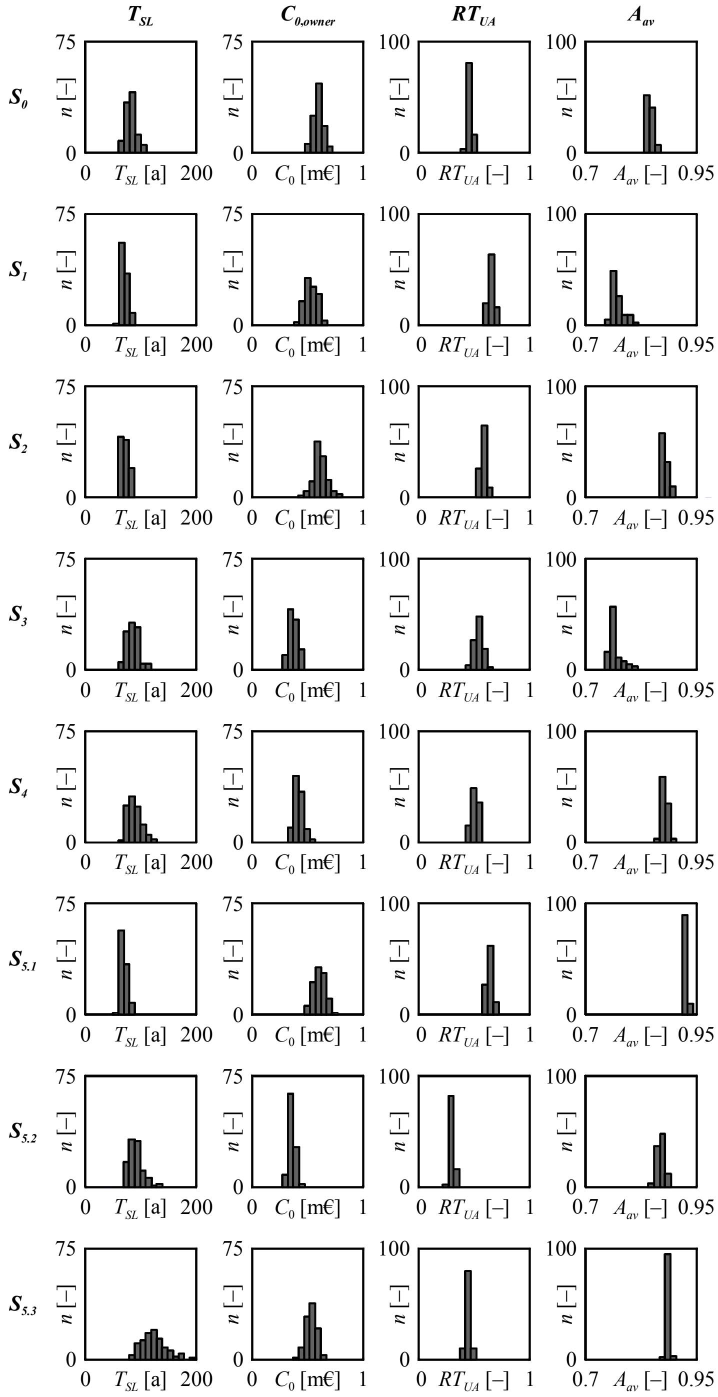



**Fig. 5.45:** Resulting distribution functions of the performance indicators for a sample size $N$=100 (Brücke Bahrenfelder Chaussee)

### 5.7.11 Discussion of the Simulation Results

**Service Life of the Köhlbrandbrücke**

The strategy $S_{5.3}$ clearly stands out with the highest mean value for the service life $T_{SL}$, which is expected to be around 131±4 years at a 95%-confidence level. The second and third highest expected service life results from the preventive-proactive strategy $S_4$ and the observed strategy $S_0$, which lead to an expected service life around 108±4 years and 99±3 years respectively. The fourth place is shared by the strategies $S_2$, $S_3$ and $S_{5.2}$, which in the limits of accuracy lead to comparable mean service lives in the range between 82 to 85 years, each with a 3-year 95%-confidence interval. Finally, the strategies $S_1$ and $S_{5.1}$ result in the lowest mean service life values, which are found to be ±2 years around 61 and 63 years, respectively.

The ultimate limit states that cause the end of service life occur for a handful of *hot spots* (Fig. 5.46), which are specific failure types—bending or shear—that occur at specific SAMS elements. A common *hot spot* for all strategies is the SAMS element '01.5', which represents the span elements of the axis range 00 to 30 of the eastern ramp (see Fig. 5.5) and is the critical element for deterioration induced bending failure. Another common weak point is found to be the SAMS element '101', which in structural terms represents the pier elements of the eastern part of the western ramp. This element becomes critical both with respect to negative bending over the pier and with respect to shear. Further critical elements, which however do not occur with the intensity of the two before mentioned *hot spots,* are found to be the SAMS-elements '57.5' and '117'. The first one structurally represents positive bending at the eastern ramp-intersection 32/36/57, as it is depicted in Fig. 5.5. The latter of the two elements represents negative bending and shear at the piers of the western part of the western ramp. Furthermore, if only exceptionally for strategy $S_3$ and $S_{5.2}$, the element '101.5' becomes critical in terms of positive bending; this element represents the span elements of the western part of the western ramp.

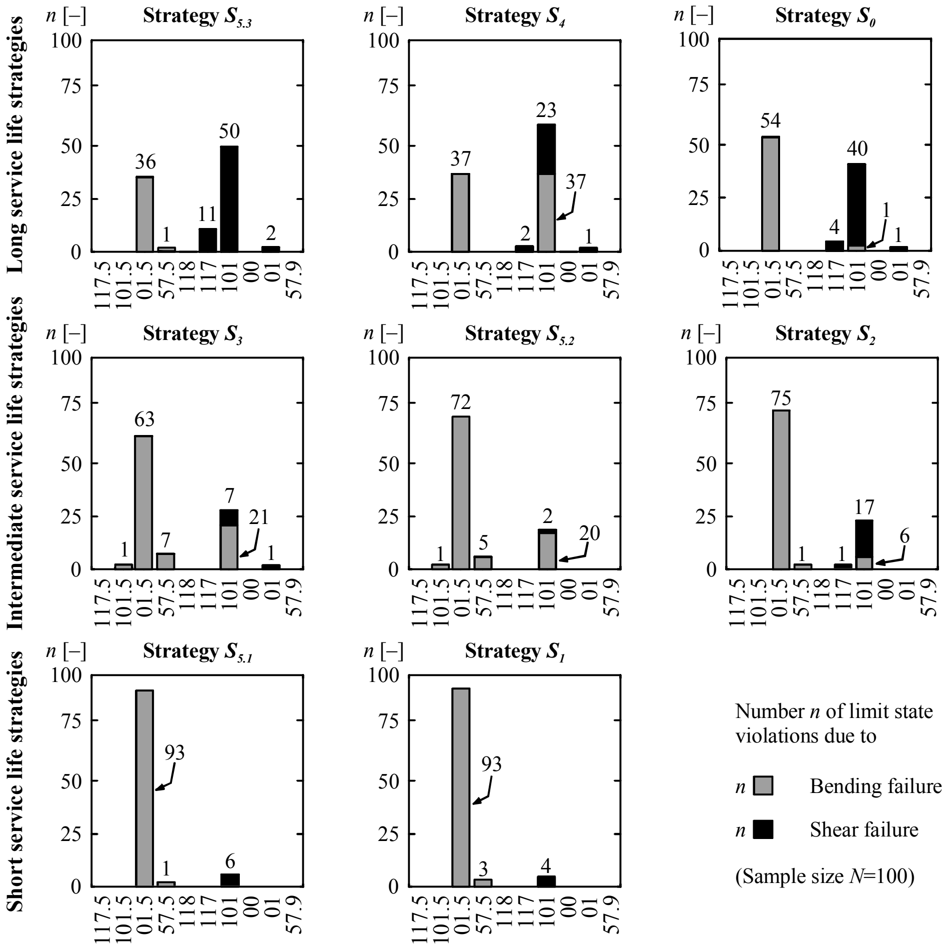



**Fig. 5.46:** Local distribution of the ultimate limit states that cause the end of service life (Köhlbrandbrücke)

The results for the short service life strategies $S_{5.1}$ and $S_1$ reveal a clear and common hot spot in the form of bending failure at the element '01.5' (see Fig. 5.46, bottom row). The reason for this is the high tendon corrosion intensity, which is assumed according to the detected tendon corrosion in the span between axis 4 and 5 (see Table 5-1). Also, the element '57.5' becomes critical in certain cases. This element also is characterised by a high tendon corrosion rate, according to the recorded corrosion defects at the *Breslauer Rampe* between axis 50 and 52 (Table 5-1). However, because of the considerably high initial safety level of around 1.6, compared to a value of around 1.18 for the element '01.5', it only exceptionally becomes critical when the corrosion rates at element '57.5' is considerably higher compared to '01.5' (see Fig. 5.47 (a) and (b)). Accordingly, the SAMS elements '101.5' and '101' with their considerably lower deterioration rates have not been found critical, even if they initially show lower safety levels compared to '01,5'.

The exceptionally occurring shear failure at the element '101' can be only found for sample elements with very high shear reinforcement corrosion and low effects of the maintenance activities, such as shear reinforcement conservation. As it is explained above in sub-section 5.7.2, the remarkably low strengthening effect that occurs at axis '101' compared to the other locations, such as '00' or '57.9', results from the fact that axis '101' represents the location with the highest shear and the highest shear reinforcement ratio along the whole bridge so that the relative strengthening effect is lower compared to the other locations.

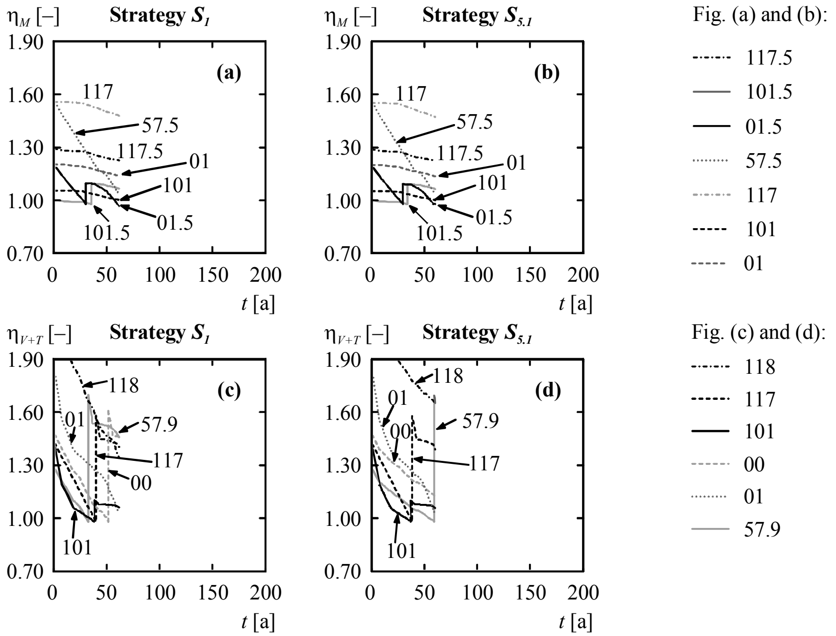



**Fig. 5.47:** Development of the degrees of fulfilment for bending and shear in the case of Strategy $S_1$ and $S_{5.1}$ (Köhlbrandbrücke)

In the case of the *intermediate service life strategies* $S_2$, $S_3$, and $S_{5.2}$, for which the detailed η-developments are depicted in Fig. 5.48, the predominant lifetime limiting factor is still found to be the bending capacity in the spans of the eastern ramp in the range from axis 00 to 30, as it is represented by element '01.5' (see Fig. 5.46). However, it can be seen in the middle row of Fig. 5.46 that—compared to the *low service life strategies* ($S_1$ and $S_{5.1}$) in the bottom row—a shift occurs in terms of absolute failure numbers from the element '01.5' to '101'. In the case of the proactive strategy $S_2$, this shift mainly occurs in terms of more frequent shear failure at the pier in axis 101. This can be explained in most parts by the strategy specific procedure of applying tendon conservation works together with girder concrete repair works. By doing so, the tendon deterioration progress at the critical element ‘01.5’ is considerably delayed, as can be seen from comparing Fig. 5.48 (a) with Fig. 5.47 (a). However, the strategy specific delay in the deterioration progress at the shear reinforcement of the element ‘101’ falls back behind the above described delay in tendon corrosion. As a result, the service life limit following from shear reinforcement corrosion at ‘101’ comes close to the lifetime limit with respect to bending failure at ‘01.5’, which is around 82 years in the case of the representative data-combination (Fig. 5.48 (a) and (d)); accordingly, shear failure can become the service life limiting factor more often. Also, bending failure at ‘101’ can become critical for certain cases, because the degradation at this location is almost unaffected by the strategy. Thus, bending failure at axis ‘101’ can compete with the prolonged time to a critical bending state at ’01.5’.

In contrast to this, both the *preventive repair strategy* $S_3$ and the *appearance focused strategy* $S_{5.2}$ do not exhibit such increased numbers of shear failure. This is because both strategies apply a preventive surface treatment, either in the form of hydrophobic impregnation ($S_3$) or coating ($S_{5.2}$). This effectively delays shear corrosion similarly to tendon corrosion so that no shift occurs from bending failure at ’01.5’ to shear failure at ‘101’. In the case of strategy $S_{5.2}$, the number of shear failures is even reduced to only two sample elements. This can be explained by the earlier concrete repair works, which in effect considerably reduce shear reinforcement corrosion. Rather, a shift for bending failure from axis ’01.5’ to ‘101’ occurs for both strategies because of the fact that the beneficial effects of preventive concrete impregnation do not apply to the tendons in the high points at axis ‘101’, where deterioration is caused by chloride attack through a leaking asphalt layer. Therefore, these two strategies practically do not affect tendon deterioration in the highpoints and thus bending failure at axis ‘101’ puts a limit to the prolonged time to a critical bending state at ’01.5’.

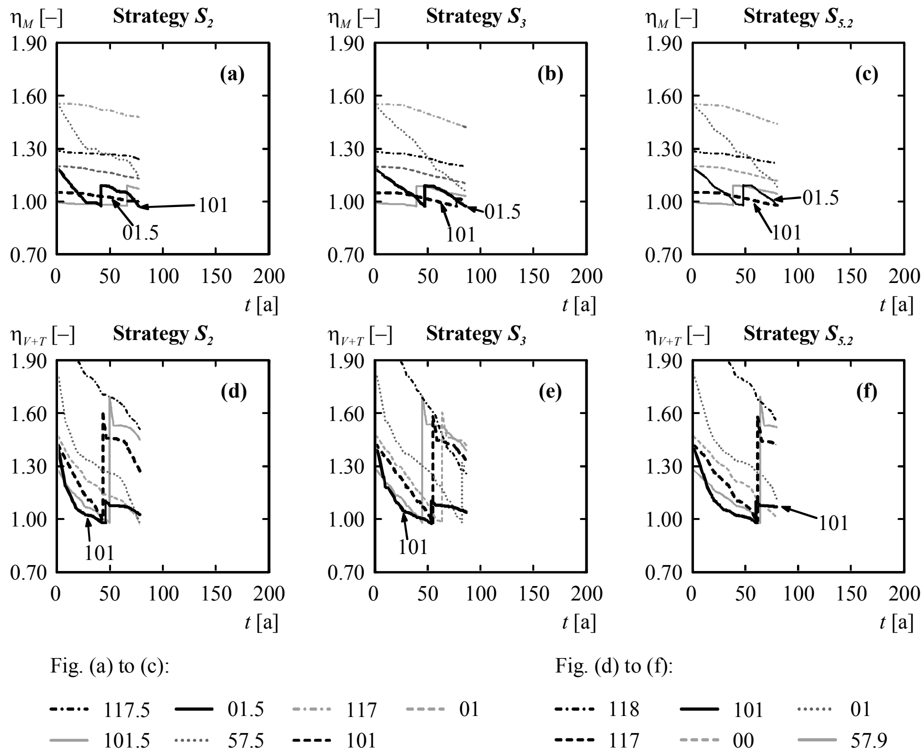



**Fig. 5.48:** Development of the degrees of fulfilment for bending and shear in the case of Strategy $S_2$, $S_3$, and $S_{5.2}$ (Köhlbrandbrücke)

Finally, the *long service life strategies* $S_0$, $S_4$ and $S_{5.3}$, for which the detailed η-developments are depicted in Fig. 5.49, show a substantial shift from bending failure at the element '01.5' to failure at the element '101' (see Fig. 5.46, top row). Remarkably, shear failure at '101' nearly becomes at par with the bending failure at '01.5' for the *observed strategy* $S_0$ or even constitutes the dominating failure mode in the case of the strategy $S_{5.3}$. At the same time, bending failure is practically irrelevant for the tendons at the highpoints of element '101' for these two strategies. In contrast to this, bending failure over the pier of axis 101 occurs in 37% of the cases for the *preventive-proactive strategy* $S_4$. The shift in the specific failure mode distribution for the strategy $S_4$ can be easily explained with reference to the above discussion of the proactive strategy $S_2$ and the preventive strategy $S_3$ because it is defined as a combination of these two strategies. Firstly, as described above for strategy $S_3$, the bending failure at '101' is found to compete with bending failure at '01.5' because of the fact that the beneficial effects of preventive concrete impregnation do not apply to the tendons in the high points at axis '101'. Secondly, the temporary halt in tendon corrosion due to proactive tendon conservation prolongs the time to bending failure at '01.5' so that it comes close to the shear failure times at '101'. As a result, also shear failure can become the limiting factor in competition to bending failure by accounting for 23% of the observed failures.

The two strategies $S_0$ and $S_{5.3}$ both practically exclude bending failure over the piers by applying deck reconstruction, which leads to a long term conservation of the high positioned tendons. Therefore, bending failure at '101' does virtually not occur for these two strategies over the time horizon of the SAMS simulation; there is only one exception in the case of strategy $S_0$. Also, these two strategies are defined by intensive proactive tendon conservation works, which lead to a considerable reduction in tendon corrosion. As a result, shear failure becomes a highly relevant failure mode. In the case of the strategy $S_{5.3}$, which aims at a long structural service life and applies very intensive proactive structural interventions together with preventive hydrophobic impregnation, the above described effects are more pronounced than in the case of the other strategies.

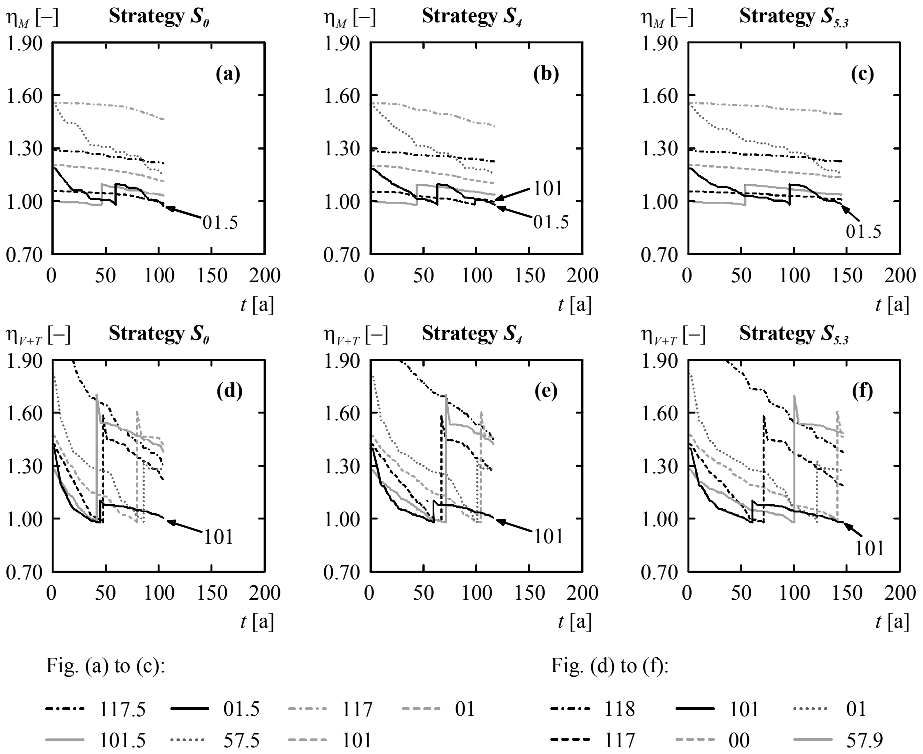



**Fig. 5.49:** Development of the degrees of fulfilment for bending and shear in the case of Strategy $S_0$, $S_4$ and $S_{5.3}$ (Köhlbrandbrücke)

**Service Life of the Brücke Bahrenfelder Chaussee**

In the case of the *Brücke Bahrenfelder Chaussee* the best outcome, as intended, results from the *structural service life focus strategy* $S_{5.3}$, of which the application delivers a sample mean value of 187 years. Good service life results are also achieved by applying the *preventive-proactive strategy $S_4$* ($\mu$=134 years) and the *appearance focus strategy $S_{5.2}$* with sample mean service life of 138 years. Medium service life results are achieved by the *proactive strategy $S_2$* ($\mu$=109 years), the *really applied strategy $S_0$* ($\mu$=124 years), and the *preventive repair strategy $S_3$* ($\mu$=128 years). The lowest service life results from the strategies $S_1$ and $S_{5.1}$, both of which are strictly non-preventive as well as reactive in structural terms. These two strategies result in a mean service life of 105 years.

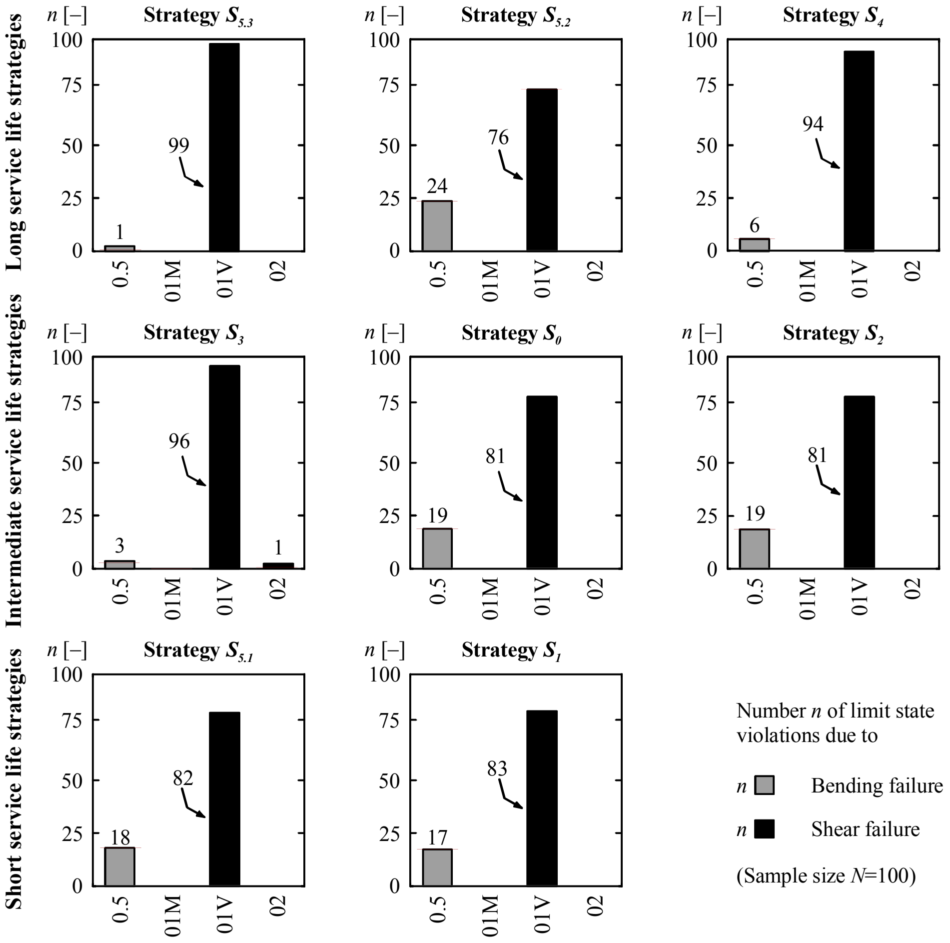



**Fig. 5.50:** Local distribution of the ultimate limit states that cause the end of service life (Brücke Bahrenfelder Chaussee)

It is found, that the ultimate limits states that cause the end of service life occur for just two *hot spots* (Fig. 5.50). The prominent *hot spot* for all strategies is the SAMS element ’01V’, which represents the shear failure at the intermediate support of the bridge. The second critical failure mode is found to be positive bending action in the western span of the bridge as it is represented by the element '0.5'. For the investigated sample of size 100 only one exception, where the failure occurs at another location, was found: For strategy $S_3$ one case of shear failure at the eastern support '02' was observed (Fig. 5.50).

For the two strategies $S_1$ and $S_{5.1}$, which lead to the lowest service life results, the main *hot spot* is found in terms of shear failure at the intermediate support. It can be seen from Fig. 5.50 that for these strategies shear failure at the element '01V' accounts for 82% (83%) of the critical limit states, while bending failure at the span element is observed only in 18% (17%) of all cases. A further data analysis showed that the more rarely occurring bending failure in the western span at '0.5' becomes the lifetime-limiting factor for sample elements with high tendon corrosion rates in the span element, as it is the case for sample element No. 9 (see Fig. 5.51 (b) and (d)).

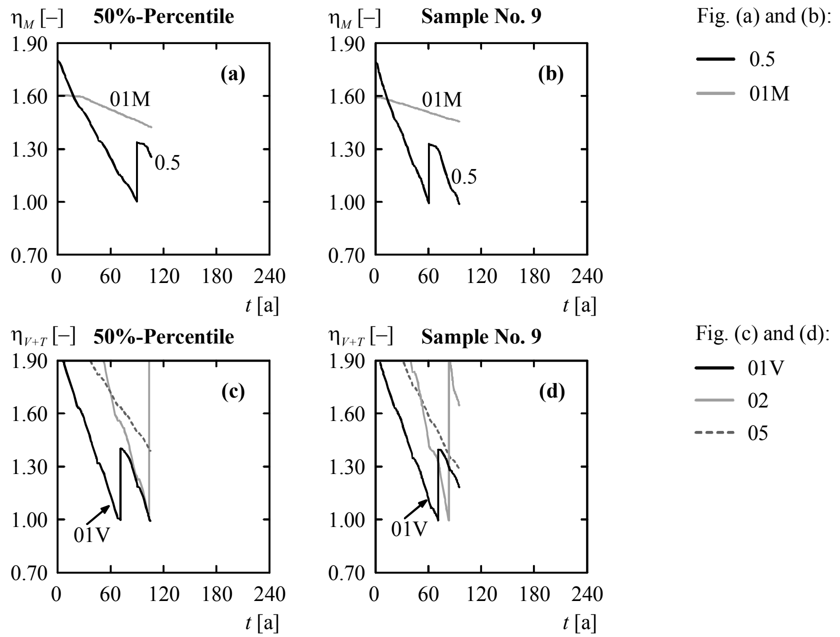



**Fig. 5.51:** Development of the degrees of fulfilment for bending and shear in the case of Strategy $S_1$ (Brücke Bahrenfelder Chaussee): (a) and (c) depict the results that are obtained from the representative parameter-value combination (i.e. 50%-percentile), where shear failure at '01V' marks the lifetime limit; (b) and (d) depict the developments that result from the sample element No. 9, where bending failure in the western spans becomes decisive, as it is the case with 17% of all sample elements.

The three strategies $S_3$, $S_0$, and $S_2$, for which the detailed η-developments are depicted in Fig. 5.52, lead to an increased service life by means of different approaches. The *proactive* concrete girder repair works of strategy $S_2$ extend the time to shear failure near the intermediate support as well the time to critical bending failure in the western span by repeatedly reducing the corrosion rates as well as the alkali-aggregate reaction processes within the web concrete, while this effect is achieved by strategy $S_3$ with the help of *preventive* cyclic hydrophobic impregnation. As it can be seen in Table 5-15 and Fig. 5.45, the *preventive* strategy $S_3$ with its $\mu$=128 years leads to significantly better results than the *proactive* approach of strategy $S_2$ ($\mu$=109 years). Although the really applied strategy $S_0$ can be described—to a certain extent—as a *preventive-proactive* strategy, which applies *proactive* concrete repair in combination with *preventive* concrete coating, it does not lead to a further increased service life compared to strategy $S_3$ but only results in a mean service life of 124 years. This is because concrete coating is not applied on a cyclical time basis, as it is the case for the preventive hydrophobic impregnation of strategy $S_3$, but is applied only together with concrete repair works. Hence, concrete repair and coating are applied in intervals of about 25 years, so that coating is not applied in the optimum cycle of 15 years and therefore periods of ineffective coating occur between subsequent treatments. Furthermore, concrete repair is not applied as often as in the case of strategy $S_2$. As a result, the rates for shear reinforcement corrosion and alkali-aggregate reaction can be found to lie between those of the two strategies $S_2$ and $S_3$.

It must be kept mind that for the illustrative parameter-value combination shear failure at the element ‘01V’ constitutes the lifetime limiting factor for each of the three strategies. However, as can be seen from Fig. 5.50, also bending failure in the western span can become critical for sample elements with high tendon corrosion rates and comparatively lower shear reinforcement corrosion rates. In the case of strategy $S_2$ and $S_0$ this occurs for 19% of the investigated sample elements; however, in the case of the *preventive strategy* $S_3$ the share of bending failure is reduced to 3% (Fig. 5.50). This sharp fall in bending failure happens because of the significant reduction of the tendon corrosion rate by hydrophobic impregnation.

Also, exceptional shear failure at the eastern support ‘02’ in the case of the *preventive strategy* $S_3$ is found for sample element No. 43. As can be seen in Fig. 5.52 (g)–(i), a specific parameter value constellation occurs for this sample element: A considerably high shear reinforcement corrosion rate at the outer support, which exceeds the rate at the intermediate support by around 40%, is combined with a similarly lower effect of the hydrophobic impregnation on the corrosion progress at that location with a potentially leaking expansion joint. This specific constellation causes the shear reinforcement corrosion curve of the element ‘02’ to overtake the corrosion curve of element ‘01V’. However, this does not happen in the case of strategy $S_0$ because the higher corrosion rate at the eastern support ‘02’ is compensated by a significantly higher beneficial effect of reinforcement conservation and more frequent expansion joint maintenance, so that in effect still shear failure at ‘01V’ becomes the lifetime limiting factor. Similarly, the proactive strategy $S_2$ with more frequent expansion joint maintenance shows lower resulting shear resistance deterioration rates at ‘02’.

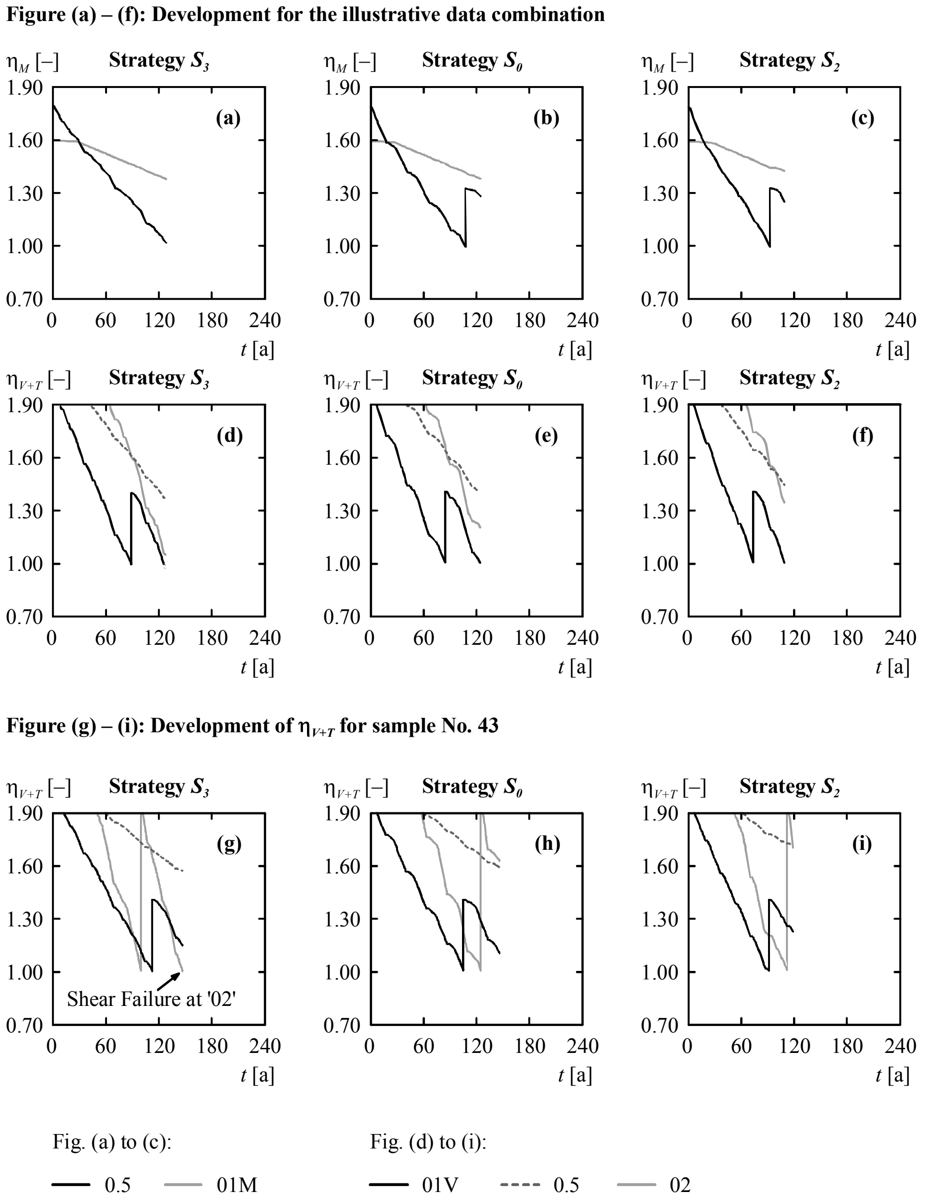



**Fig. 5.52:** Development of the degrees of fulfilment for bending and shear in the case of Strategy $S_3$, $S_0$, and $S_2$ (Brücke Bahrenfelder Chaussee)

Further service life improvement can be achieved by applying the strategies $S_4$, $S_{5.2}$, and $S_{5.3}$, for which the detailed η-developments are depicted in Fig. 5.53. The *preventive-proactive strategy $S_4$* simply combines the above described two approaches for reducing the rates of tendon and shear reinforcement corrosion: The combination of proactive concrete repair and cyclic hydrophobic impregnation significantly extends the time to ultimate structural failure to a sample mean value of 134 years (Table 5-15). An analogous approach is implemented by strategy $S_{5.2}$, which leads to even better service life results (μ=138 years) because of the more frequent concrete repair works and the higher effect of coating on tendon and shear reinforcement corrosion. Finally, the best service life results for the *structural focus strategy* $S_{5.3}$ with a sample mean value of 187 years. This result is achieved by systematically reducing the tendon and shear reinforcement corrosion rates by more intensive concrete repair works, tendon and reinforcement conservation, and hydrophobic impregnation (Fig. 5.53)

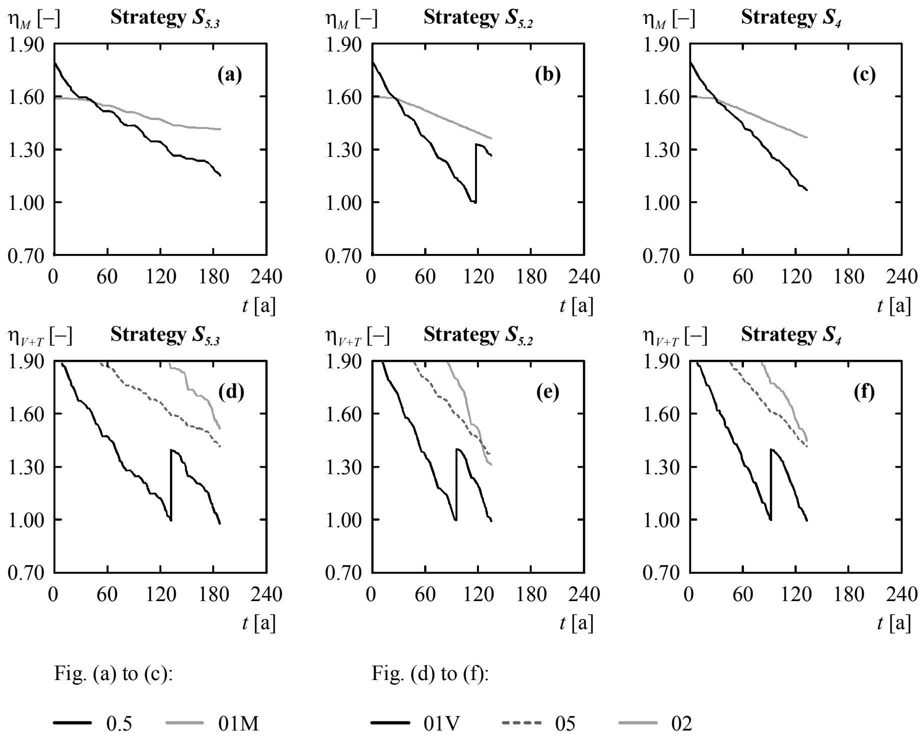



**Fig. 5.53:** Development of the degrees of fulfilment for bending and shear in the case of Strategy $S_4$, $S_{5.2}$, and $S_{5.3}$ (Brücke Bahrenfelder Chaussee)

**Economic Results for the Köhlbrandbrücke**

As can be seen in Table 5-14, the best economic results in terms of the net present value of the owner's costs ($C_{0,owner}$) are provided by the *preventive strategy* $S_3$, the *preventive-proactive strategy* $S_4$, and the *appearance-focus strategy* $S_{5.2}$. These strategies lead to sample mean values in the range from 12.7 million € to 13.1 million € for the discounted costs $C_{0,owner}$. The second group consists of the strategies $S_0$ and the structural condition-focus strategy $S_{5.3}$ which result in an almost identical mean value of the life cycle costs of around 15.7 million €. The bottom group consists of the proactive strategy $S_2$, the reactive strategy $S_1$ and the traffic-centred strategy $S_{5.1}$. Here strategy $S_2$ an $S_1$ show comparable results around 17.6 to 18.3 million € while the strategy $S_{5.1}$ clearly stands out as the most expensive strategy with a mean value for the discounted costs of around 20.6 million €.

The following Fig. 5.54 illustrates for each strategy the maintenance and replacement costs $C_{maint}$ for each year and their contribution to the net present value of all costs $C_{0,owner}$ ($t$). In this figure, the continuous curve refers to the left hand scale and represents the indicator $C_{0,owner}$ ($t$) from the beginning to the actual year. In turn, the bar-chart, which refers to the right hand scale, illustrates the costs $C_{maint}$ for each year. Consequently, the $C_{0,owner}$ ($t$)-curve has a stair-like shape and each jump illustrates the actual contribution of the maintenance expenditure for each year on the cumulated discounted cost. The limit to which the $C_{0,owner}$ ($t$)-curve is tending with more and more life-cycles, is in fact the economic performance indicator $C_{0,owner}$ that refers to an unlimited time horizon, as it is explained in sub-section 4.7.4 of the present thesis. It must be kept in mind that Fig. 5.54 depicts the cost profiles as they result for the representative parameter-value combination—that is a single input value combination, where all values are set to the 50%-percentile of their respective distributions. As a consequence, the resulting $C_{0,owner}$ values in Fig. 5.54 differ from the values of Table 5-14, which in turn refer to sample mean value of a stochastic sample of 100 elements.

From Fig. 5.54 it can be seen that in the case of the worst performing strategies $S_1$ and $S_{5.1}$ the bridge replacement costs, which occur after a service life of around 61 or 63 years, considerably contribute to the high resulting value for $C_{0,owner}$. Even the second bridge replacement, which occurs after around 122 or 126, years still noticeably contributes to the owners costs. Similarly, the economic performance of the slightly better performing proactive strategy $S_2$ still suffers from a relatively short service life of about 82 years, which leads to a considerable jump in the $C_{0,owner}$-curve at the time of the first bridge replacement. Nevertheless, the approximately 20 year longer service life compared to strategies $S_1$ and $S_{5.1}$ noticeably reduces the economic effect of the bridge replacement costs and compensates the otherwise higher cost contribution of the maintenance and rehabilitation works that result from the proactive approach, which results in earlier interventions with lower discounting effects.

Similarly, the better performing strategies $S_0$ and $S_{5.3}$ clearly benefit from the even more extended service life, which significantly reduces the economic impact of future bridge replacement: In the case of strategy $S_{5.3}$ the costs for the first bridge replacement only do contribute by a mere 1% of their original value to the net present value of the life-cycle costs, compared to about 16% in the case of strategy $S_1$. Analogously to the above mentioned proactive strategy $S_2$, this effect compensates the higher effects from the maintenance costs that result from the early intervention policies, which are implemented both by strategy $S_0$ and $S_{5.3}$.

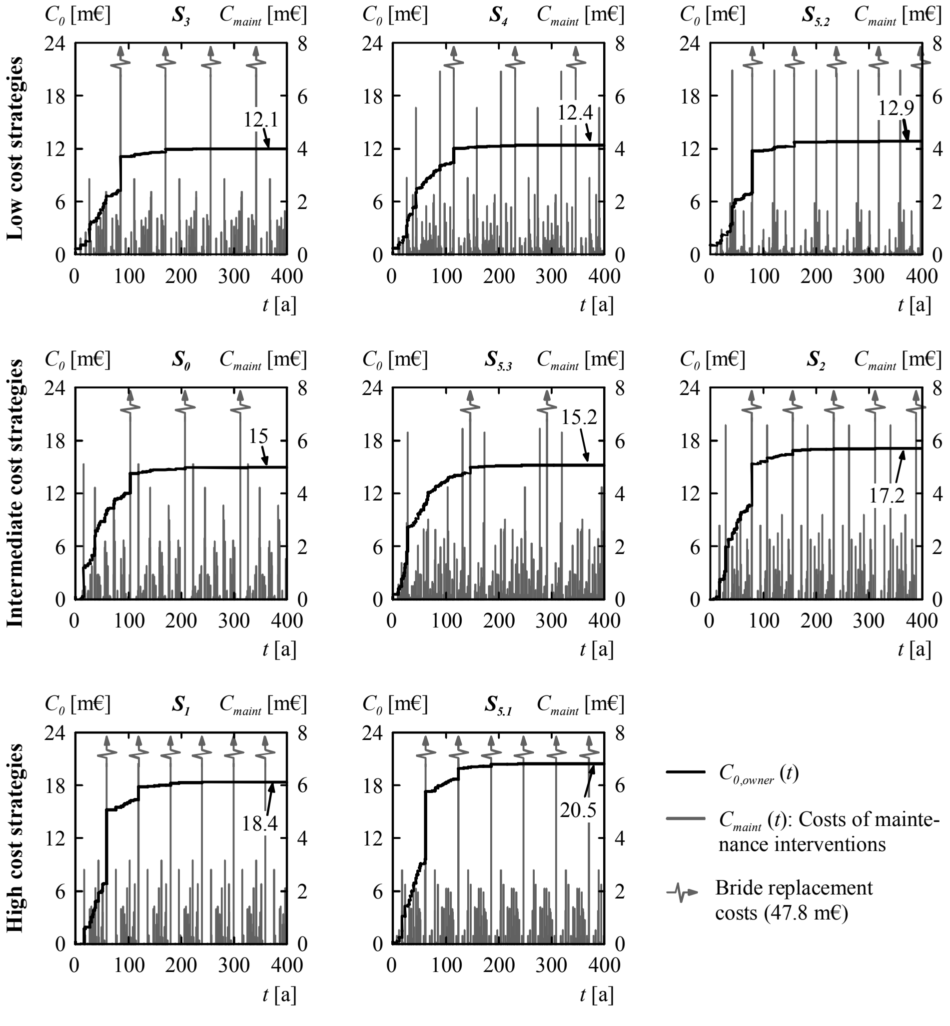



**Fig. 5.54:** Service life cost profiles of the representative parameter-value combination (50%-percentile) for the Köhlbrandbrücke.

Opposed to this, two of the top performing strategies—the appearance-focused strategy $S_{5.2}$ and the preventive repair strategy $S_3$—show a high economic performance in spite of a service life, which is in the range of the bad performing proactive repair strategy $S_2$. This can be explained by the preventive nature of both strategies, which both apply preventive concrete treatment—concrete coating in the case of $S_{5.2}$ and hydrophobic impregnation in the case of $S_3$—and thus delay the cost-intensive concrete rehabilitation works; in the case of strategy $S_{5.2}$ the routinely applied coating even completely prevents girder concrete repair works. The preventive repair strategy $S_4$ combines the two positive effects of a longer service life and a preventive approach. However, because the longer service life also results from the proactive approach with its adverse economic effect of earlier maintenance application, it is not the best performing strategy.

**Economic Results for the Brücke Bahrenfelder Chaussee**

The best economic results, as can be seen in Table 5-15, are provided by the *appearance-focus strategy* $S_{5.2}$, which exceeds all investigated strategies by delivering a sample mean value for $C_{0,owner}$ of 363 k€. It is closely followed by the *preventive strategy* $S_3$ with an expected value of 375 k€. The second ranking group consists of the strategies $S_4$, $S_1$, and $S_{5.3}$, which result in mean value of the life cycle costs in the range from 424 k€ ($S_4$) to 539 k€ ($S_{5.3}$). The worst economic results in terms of the net present value of the owner's costs $C_{0,owner}$ are provided by the *really applied strategy* $S_0$, the *traffic-focus strategy* $S_{5.1}$, and the proactive strategy $S_2$. As can be seen from Table 5-15, these strategies lead to discounted costs $C_{0,owner}$ in the range from 594 k€ ($S_{5.1}$) to 626 k€ ($S_2$).

Analogously to the above discussion for the Köhlbrandbrücke, the following Fig. 5.55 illustrates for each strategy the yearly maintenance costs $C_{maint}$ and their contribution to the net present value of all costs $C_{0,owner}(t)$. From this Fig. 5.55 it can be concluded that for all strategies the resulting value for $C_{0,owner}$ crucially depends on the timing of the concrete repair activities, which include *concrete repair of girders and abutments* as well as *sidewalk rehabilitation*. The girder concrete repair activity excels all other intervention activities in terms of costs—with the only exception of bridge replacement of course. Likewise, sidewalk rehabilitation also is one of the more expensive measures. Also, both activities are repeatedly applied during a bridge life cycle. In contrast, the most expensive activity *bridge replacement* only does contribute noticeably to the resulting $C_{0,owner}$-value for strategies resulting in a short service life ($S_1$, $S_{5.1}$, and $S_2$), because the discounting factor, which is expressed by the term $(1-i/100)^{-t}$, rapidly decreases with time. The contribution of the other maintenance activities to $C_{0,owner}$ can be traced in Fig. 5.55 in the form of the more or less diagonal increase between two subsequent jumps that follow from concrete repair interventions. As it can be seen there, the contribution of these maintenance interventions mostly falls back behind that of the concrete repair interventions. As it is said above, it must be kept in mind that Fig. 5.55 depicts the cost profiles as they result for the illustrative parameter-value combination and thus the final $C_{0,owner}$–value differs from the sample mean values given in Table 5-15.

The worst economic results—with a $C_{0,owner}$-value of 626 k€ according to Table 5-15—are found for the *proactive strategy* $S_2$. This can be easily explained from the fact that this strategy leads to a concrete repair cycle of less than 20 years because of the proactive threshold for concrete repair of $CI_{CS,res}$=0.375 and because of the fact that concrete deterioration is not delayed by any measures, such as impregnation or coating. This 20 year repair cycle turns out to be the shortest one that is found for all investigated strategies and it is decisive for the poor economic performance. Also the poor $T_{SL}$ result with a sample mean value of only 109 years leads to a considerable contribution of the replacement costs to the $C_{0,owner}$-value, as it appears in the form of the steep increase at the end of the first service-life cycle in Fig. 5.55.

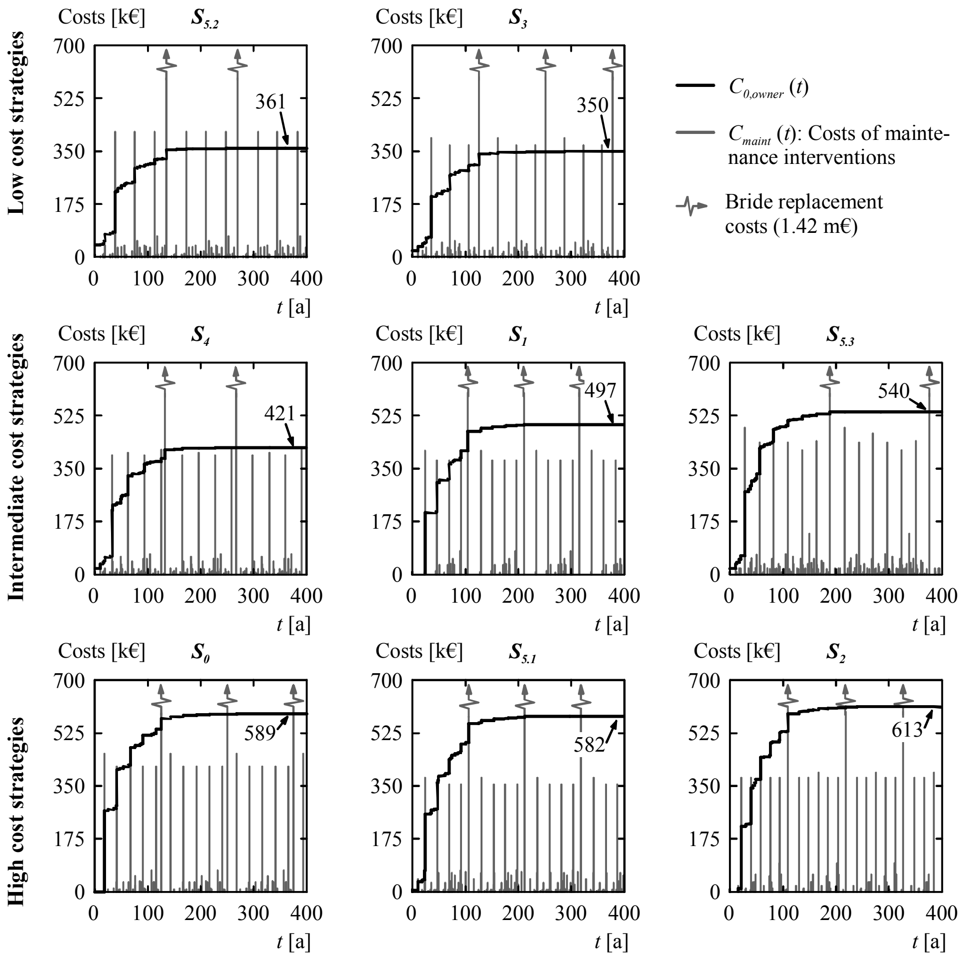



**Fig. 5.55:** Service life cost profiles (Brücke Bahrenfelder Chaussee)

At the same time, the low performing *traffic focus strategy* $S_{5.1}$ has a longer concrete repair cycle of 24 years. However, both the short pavement replacement cycle and the expansion joint maintenance cycle that characterise this strategy are acting as significant cost drivers. Moreover, the poor $T_{SL}$ results with a sample mean value of only 105 years pose a noticeable negative effect in terms of a high net present value of the replacement costs.

The *really applied strategy* $S_0$ also leads to a relatively short concrete repair cycle of around 24 years in spite of a proactive intervention threshold of $CI_{CS,res}$=0.47 compared to the reactive threshold $CI_{CS,res}$=0.0 of strategy $S_{5.1}$ which does apply concrete repair only reactively at critical deterioration levels. This 24 year repair cycle of strategy $S_0$ follows from the strategy-specific application of concrete coating and its deterioration delaying effect. Compared to strategy $S_{5.1}$ and $S_2$ it does lead to better economic results because the additional costs for concrete coating are offset by the beneficial time-effects of longer maintenance and replacement cycles with respect to the concrete surface itself and also with respect to asphalt pavement and the expansion joints, both of which are maintained according to much softer threshold values.

The strategies $S_{5.3}$, $S_1$, and $S_4$, which constitute the second ranking group with respect to the economic performance, show different concrete repair cycle times ranging from 23 years for the *reactive strategy* $S_1$ to 28 and 31 years in the case of the *proactive* strategies $S_{5.3}$ and $S_4$. The highly proactive *structural service life focus strategy* $S_{5.3}$ exhibits right from the beginning an extraordinarily dense intervention schedule, which leads to a steep increase in the $C_{0,owner}$-value already during the first years of the bridge service life. Also later on, the highly proactive intervention regime leads to a steep increase in $C_{0,owner}$ between the prominent jumps that follow from the cost-intensive concrete repair works and the associated tendon and shear reinforcement conservation works. However, the repeatedly applied preventive hydrophobic impregnation not only stretches the concrete repair cycle to 28 years but also—in combination with the other proactive maintenance intervention works—considerably extends the service life. As a consequence, the cost impact of the main influencing activities—concrete repair and reconstruction—is considerably reduced. This beneficial effect overbalances the cost impact of the before mentioned dense intervention schedule. Similarly, the *preventive-proactive strategy* $S_4$ delivers economic benefits from the even further extended concrete repair cycle of 31 years. Also its less thorough proactive maintenance regime in structural terms—mainly in terms of no automatic tendon and shear reinforcement conservation works together with concrete repair—further reduces the economic impact. As a result, the preventive-proactive strategy ranks highest within the second group. In contrast to this, the purely *reactive strategy* $S_1$ does not benefit economically from extended concrete repair cycles, in fact it leads to a short cycle of 23 years. Yet, it takes advantage from the fact that each intervention is applied as late as possible, so that the beneficial time-effects of discounting are fully exploited: Under the bridge specific constellation of input parameters, during the first 46 years of service life virtually no maintenance interventions take place—apart from the two concrete repair interventions after 23 and 46 years. The only exception is found in the form of each one repair intervention with respect to the bearings and the expansion joint, both of which belong to the cheapest of all considered maintenance options. Moreover, the reactive approach has the effect that only the urgently needed activities are applied and that practically all maintenance activities other than concrete repair take place during the twenty-year period between 60 and 80 years after construction and thus are subject to substantial discounting effects. By means of this intervention regime, this strategy is well positioned in economic terms between the two other strategies $S_4$ and $S_{5.3}$ within the second ranking group.

The two best performing strategies $S_{5.2}$ and $S_3$ draw on the beneficial effects of preventive impregnation or concrete coating, which both significantly extend the concrete repair cycles. The absolutely best performing *preventive strategy* $S_{5.2}$ results in a concrete repair cycle of approximately 37 years by applying very effective and durable concrete coating. Further, if not that significant, economic advantages follow from the reactive maintenance regime with respect to the bearings and the drainage system, which reduces the impact of the discounted maintenance expenses. Also the very well performing *preventive strategy* $S_3$ combines the positive economic effects of the preventive concrete treatment and of the reactive approach to the bridge equipment. However, due to the less effective hydrophobic impregnation works, only a shorter concrete repair cycle of 35 years can be reached.

**Visual Appearance of the Köhlbrandbrücke**

A clearly superior aesthetic performance—with a sample mean value of $RT_{UA}$=0.24 (Table 5-14)—is achieved by applying the strategy $S_{5.2}$, whose very aim is to provide for a good aesthetic appearance of the bridge. Also the strategies $S_{5.3}$ and $S_4$ show very good results around 0.44 and 0.56, respectively. Medium performance results in the range from $RT_{UA}$=0.61 to 0.71 follow from strategies $S_2$, $S_3$, and $S_0$, while the strategies $S_1$ and $S_{5.1}$ lead to the worst results with an expected mean value of 0.79.

Both the poor performing strategies $S_1$ and $S_{5.1}$ are strictly reactive with respect to concrete surface maintenance works and thus these strategies allow long periods with visible damage. Specifically, concrete deterioration levels with adverse effects on the aesthetic appearance occur over 79% of the service life on average.

A slight improvement of the aesthetic performance to a mean $RT_{UA}$-value of around 70% can be achieved by applying the strategy $S_0$ or the preventive repair strategy $S_3$. These two strategies exemplify two different approaches for achieving a better aesthetic performance. The first one, as it is applied by the preventive repair strategy $S_3$, is to apply hydrophobic impregnation in order to delay concrete surface degradation. As can be seen in Fig. 5.56, the improvement in the aesthetic performance, compared to the strategies $S_1$ and $S_{5.1}$, simply results from the extended initial period after bridge construction where visible surface defects are yet to develop. An alternative method is found by introducing a proactive concrete repair regime as it is applied by strategy $S_0$. This strategy follows a proactive concrete maintenance scheme with girder repair along the entire bridge at a threshold value of $CI_{CS,res}$=0.2 and early sidewalk repair at individual ramps. As can be seen in Fig. 5.56, the improvement in the aesthetic performance, compared to the strategies $S_1$ and $S_{5.1}$ results from the girder repair works around 36 years after construction, which take place earlier than for these two poor performing strategies and—crucially—are applied for both ramps together so that the intervention leads to a $CI_{CS,res}$-value of close to 1 for the whole bridge.

Further successive elaborations of these improvement approaches are found in the remaining strategies $S_2$, $S_4$, $S_{5.3}$, and $S_{5.2}$. The *proactive strategy* $S_2$ simply applies a stricter threshold compared to the above described strategy $S_0$ and results in a reduction of $RT_{UA}$ to 71%. A further improvement results by applying a proactive regime in combination with preventive hydrophobic impregnation in terms of the strategy $S_4$ ($RT_{UA}$=0.56). The best performing strategies $S_{5.3}$ and $S_{5.2}$ can be described as even stricter versions of preventive-proactive concrete maintenance approaches. Here, the *aesthetic-focus* strategy $S_{5.2}$ applies highly effective concrete coating, which beneath its delaying effect on concrete deterioration also results in an instant improvement of the appearance to the as-new condition $CI_{cs}$=1.0 at the treated concrete surfaces.

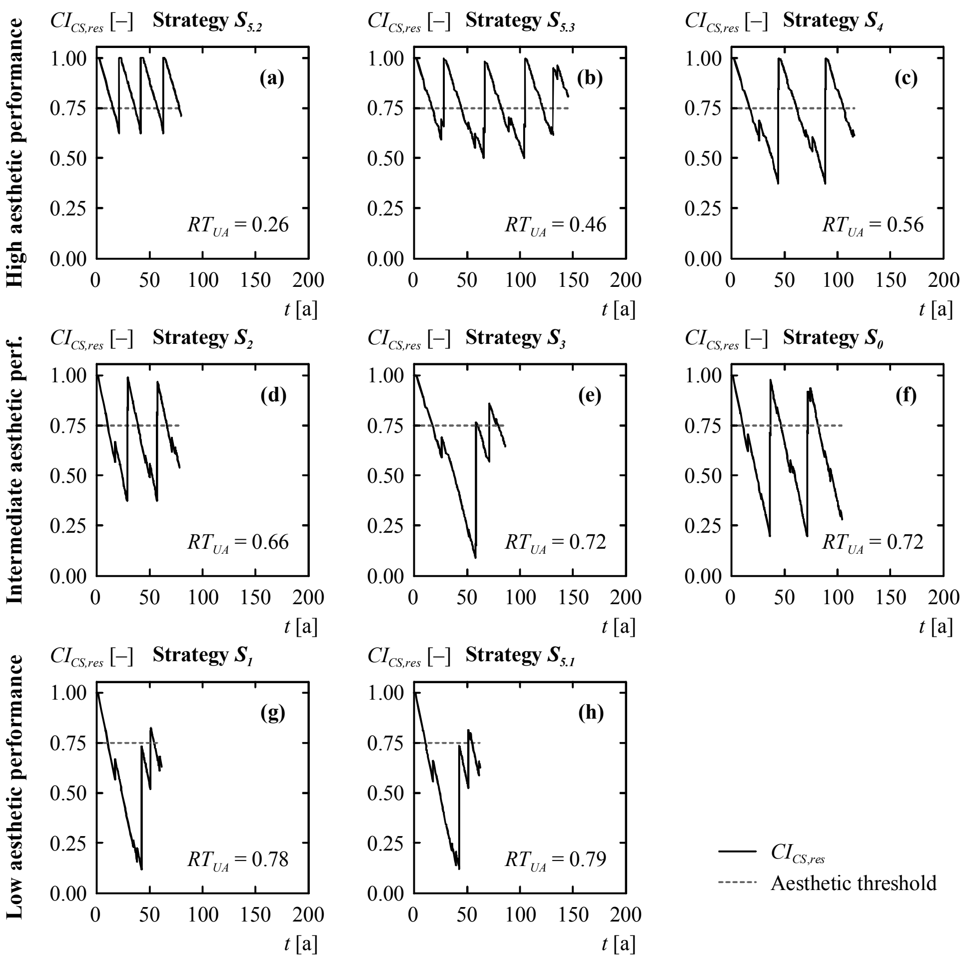



**Fig. 5.56:** Visual appearance (Köhlbrandbrücke)

**Visual Appearance of the Brücke Bahrenfelder Chaussee**

Also in the case of the Bahrenfelder Bridge, the appearance focus strategy $S_{5.2}$ turns out to be successfully implemented, as it shows superior results with an expected indicator value of $RT_{UA}$=0.31. Also the application of strategies $S_0$ and $S_4$ promises a good aesthetic appearance with $RT_{UA}$-values around 0.46 and 0.51. In the medium performance range, the strategies $S_{5.3}$, $S_3$, and $S_2$ lead to results, ranging from $RT_{UA}$ values of 0.51 to 0.59. Finally, the strategies $S_1$ and $S_{5.1}$ lead to the worst results with an expected mean value of 0.64 to 0.65.

Both poor performing strategies $S_1$ and $S_{5.1}$ are strictly reactive with respect to concrete surface maintenance works. Concrete deterioration levels with adverse effects on the aesthetic appearance occur, on average, over 65% of the service life.

An improvement of the aesthetic performance can be achieved by applying the *proactive strategy* $S_2$ or the *preventive repair strategy* $S_3$, which lead to sample mean $RT_{UA}$-values of 59% or 55%, respectively. These two strategies exemplify two different approaches for achieving a better aesthetic performance. The *preventive repair strategy* $S_3$ delays concrete deterioration with the help of hydrophobic impregnation. As can be seen in Fig. 5.57 (e), the improvement in the aesthetic performance, compared to the strategies $S_1$ and $S_{5.1}$, simply results from the fact that two of the three hydrophobic treatments that occur between subsequent concrete repair interventions take place above an overall concrete surface condition of $CI_{CS,res}$=0.75, which is used as the benchmark for calculating the $RT_{UA}$-value. Thus, the periods with a concrete surface condition above this value are more extended than the periods below that condition level. As a consequence, hydrophobic impregnation leads to a reduction in the $RT_{UA}$-value. An alternative method is found by introducing a proactive concrete repair regime as it is applied by strategy $S_2$. This strategy follows a proactive concrete maintenance scheme with girder repair along the entire bridge at a threshold value of $CI_{CS,res}$=0.375 and early sidewalk repair. As can be seen in Fig. 5.57 (f), the improvement in the aesthetic performance, compared to the strategies $S_1$ and $S_{5.1}$ simply results from reducing the time periods with $CI_{CS,res}$-values below the aesthetic threshold of 0.375 by earlier applying concrete repair works that restore a sound concrete surface.

Further successive elaborations of these improvement approaches are found in the remaining strategies $S_4$, $S_0$, $S_{5.3}$, and $S_{5.2}$. The *preventive-proactive strategy* $S_4$ simply combines the above described two approaches by following a proactive regime in combination with preventive hydrophobic impregnation and results in an improved sample mean $RT_{UA}$-value of 0.51. Also the *structural service life focus strategy* $S_{5.3}$ and the *really applied strategy* $S_0$ implement such combined approaches. With a sample mean value of $RT_{UA}$=0.51, the *strategy* $S_{5.3}$ delivers similar results compared to the above describes preventive-proactive strategy $S_4$. *The really applied strategy* $S_0$ delivers a comparable aesthetic performance with its only slightly less strict intervention threshold value $CI_{cs,res}$=0.47 and the preventive coating of the concrete surface. Finally, the *appearance focus strategy* $S_{5.2}$ yields the best aesthetic performance with the concrete surface being in unfavourable condition for only 31% of the bridge service life. This is achieved by applying a strict application threshold of $CI_{CS,res}$=0.625 and preventive hydrophobic impregnation.

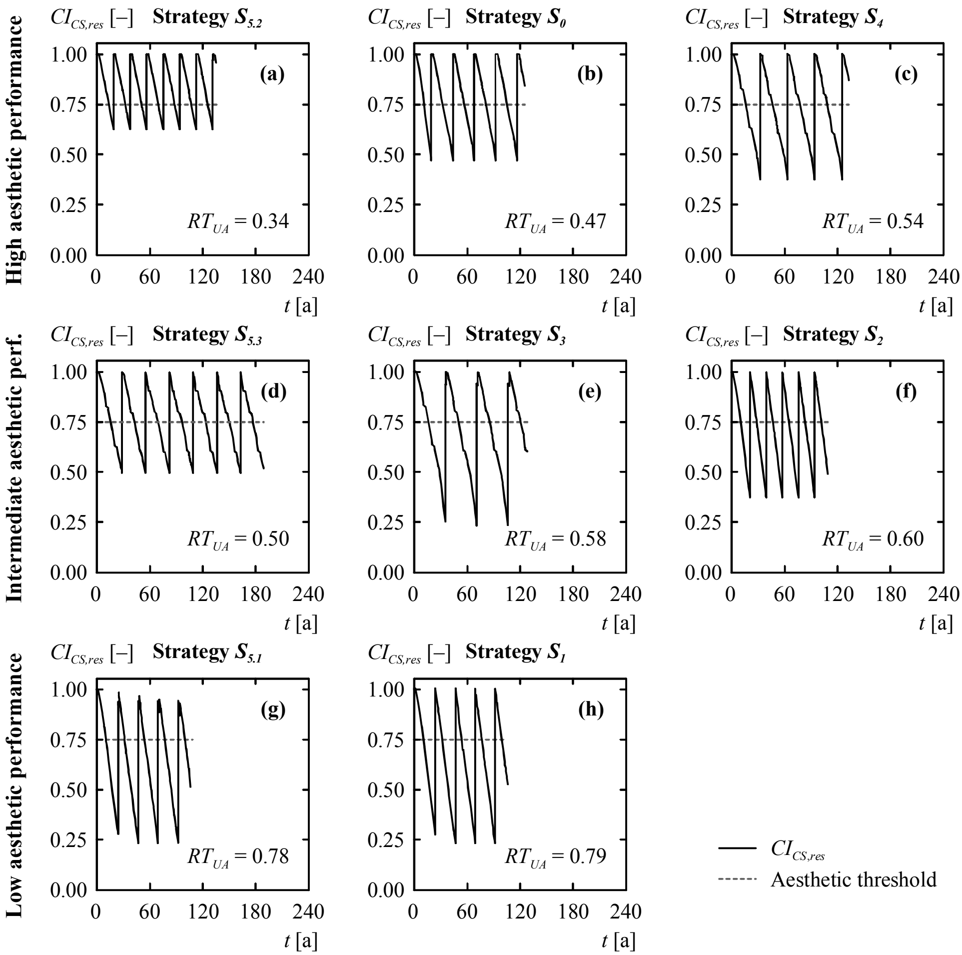



**Fig. 5.57:** Visual appearance (Bahrenfelder Chaussee)

### Traffic Availability of the Köhlbrandbrücke

From the simulation results (Table 5-14) it can be seen that the traffic-focus strategy $S_{5.1}$ is successful in leading to the highest average traffic availability, with a sample mean value of 89%. It is followed by the strategies $S_{5.3}$ and $S_{5.2}$, which also result in a high traffic performance with sample mean values of $A_{av}$=0.85 and 0.84, respectively. Slightly less availability values, ranging from 0.83 to 0.80, result from strategies $S_2$, $S_4$, and $S_0$. The poorest traffic performance is achieved by application of the strategies $S_3$ and $S_1$ with values around 0.74.

Both the low traffic-availability strategies $S_3$ and $S_1$ do apply reactive pavement and expansion joint maintenance only at a critical level, which occurs when the worst deteriorated SAMS-element reaches a condition of $CI_A$=0 or $CI_{Exp}$=0. As can be seen from the below Fig. 5.58 (g) and (h), this results in an availability profile with only exceptional restoration of a full traffic capacity (availability $a$=1.0) during the bridge service life. A closer look reveals that the preventive strategy $S_3$ shows a slightly poorer performance compared to the strategy $S_1$. This can be explained by the fact that the cyclic impregnation works do impair traffic availability.

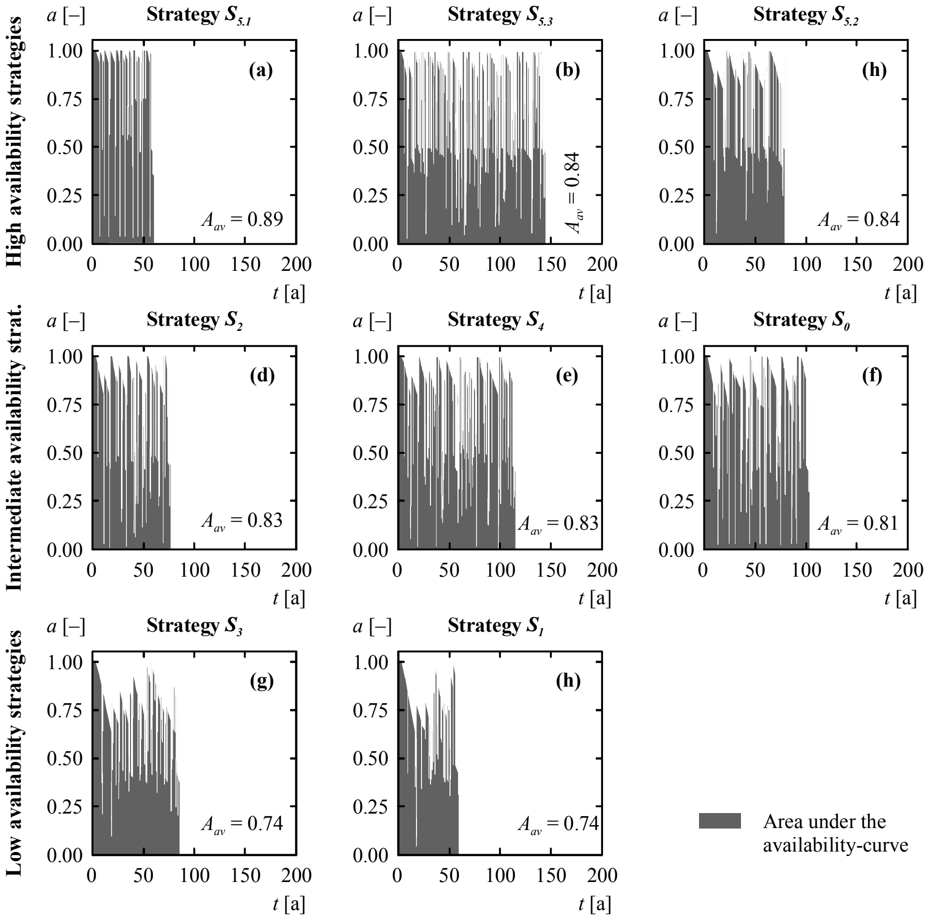



**Fig. 5.58:** Traffic availability (Köhlbrandbrücke)

A considerable improvement is found for strategy $S_0$, which applies proactive pavement replacement at alternating condition thresholds of $CI_{A,res}$=0.464 and $CI_{A,res}$=0.375 as well as proactive expansion joint repair at $CI_{Exp,res}$=0.25 and proactive expansion joint replacement at $CI_{Exp,res}$=0.53, the latter in the case that joint repair is not feasible. Logically, this proactive maintenance approach leads to a higher traffic availability by maintaining the traffic influencing condition aspects—pavement and expansion joints—at higher levels with repeated restoration of the full traffic capacity (see Fig. 5.58 (f)).

A slight further improvement by 2% to 3% is achieved by the strategies $S_{5.2}$, $S_4$, $S_2$, and $S_{5.3}$, all of which apply proactive pavement and expansion joint maintenance. Each one of these strategies leads to higher availability compared to the above described strategies mainly because of the stricter condition thresholds for expansion joint maintenance which outweighs the partially higher threshold for pavement maintenance. Accordingly, the structural focus strategy $S_{5.3}$ leads to the best traffic performance within this group because it applies the highest maintenance application thresholds.

As intended, the strategy $S_{5.1}$ results in the highest traffic performance, which again simply follows from the strictest thresholds for pavement and expansion joint maintenance application and the resulting restoration of full traffic capacity in very close time-intervals (Fig. 5.58 (a)).

**Traffic Availability of the Brücke Bahrenfelder Chaussee**

As it appears from the simulation results that are presented in Table 5-15, the traffic-focus strategy $S_{5.1}$ excels all other strategies with an average traffic availability of around 93%. With some distance, it is followed by the strategies $S_{5.3}$, $S_{5.2}$, $S_2$, and $S_4$, which result in mean traffic performance values in the range from 89% to 88%. Again, these strategies are followed with a distance of around 3% by the really applied strategy $S_0$, which results in a sample mean value of $A_{av}$=85%. As it is also the case with the above described *Köhlbrandbrücke*, the poorest traffic performance is achieved by application of the strategies $S_3$ and $S_1$. These lead to sample mean values in the range of 77%, with the preventive strategy $S_3$ exhibiting a slightly lower performance due to traffic disruption from cyclic impregnation works.

Both the reactive strategy $S_1$ and the preventive-repair strategy $S_3$ do apply pavement and expansion joint maintenance only at a critical level when the worst deteriorated SAMS-element reaches a condition of $CI_A$=0 or $CI_{Exp}$=0. That is why, as it is also the case with the above described *Köhlbrandbrücke*, these strategies only rarely return the traffic capacity to an unconfined level of $a$=1.0 (see Fig. 5.59 (g) and (h)). The preventive strategy $S_3$ shows with an average traffic availability of 76% a slightly poorer performance compared to the strategy $S_1$, for which a sample mean value of $A_{av}$=0.77 is found by the simulation study. Analogously to the *Köhlbrandbrücke*, this can be explained by the fact that the cyclic impregnation works do impair traffic availability.

A considerable improvement in the traffic availability to a sample mean value of around 85% is found for strategy $S_0$, which applies proactive pavement maintenance at a condition threshold of $CI_{A,res}$=0.25 as well as proactive expansion joint replacement at $CI_{Exp,res}$=0.25 and repair at $CI_{Exp,res}$=0.75 in an alternating manner. Logically, this proactive maintenance approach leads to a higher traffic availability.

A slight further improvement by 2% to 4% is achieved by the strategies $S_{5.2}$, $S_4$, $S_2$, and $S_{5.3}$, and finally, the strategy $S_{5.1}$ results in the highest traffic performance ($A_{av}$=0.928). The explanations for these results are the same as for the *Köhlbrandbrücke*.

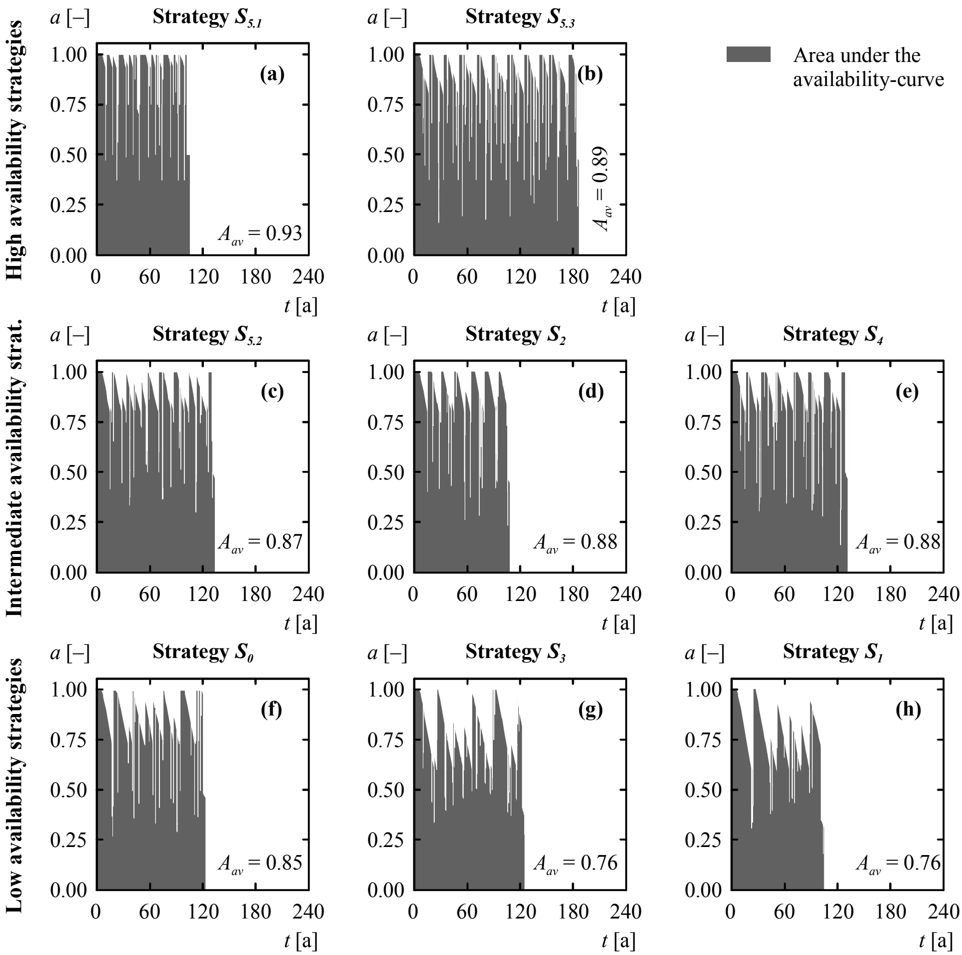



**Fig. 5.59:** Traffic availability (Bahrenfelder Chaussee)

## 5.8 Multi-Objective Strategy Evaluation

### 5.8.1 Overview

As a next step, multi-object strategy evaluation is conducted with respect to four performance aspects and their respective indicators: owner's costs ($C_{0,owner}$), service life ($T_{SL}$), traffic flow performance ($A_{av}$), and aesthetic appearance ($RT_{ua}$). An introductory investigation in sub-section 5.8.2 discusses methods for eliminating some of the strategies from further evaluation by a simple dominance analysis: On this basis it states the need for more elaborate methods beyond a simple dominance analysis approach. As a solution to this, the sub-section 5.8.3 applies the evaluation method of profile comparison. As a result, for each bridge one strategy is identified that—of all the investigated strategies—is most suitable for satisfying the bridge specific performance requirements.

### 5.8.2 Simple Performance Comparison and Dominance Analysis

The simulation results show that each strategy shows individual strength and weakness, which appears from Fig. 5.44 and Fig. 5.45, and also from the numerical values of Table 5-14 and Table 5-15. For example, in the case of both bridges the *preventive repair strategy* $S_3$ on the one hand shows excellent economic results, while this strategy only appears in the middle group with respect to visual appearance and service life; it even delivers bottom group results with respect to traffic availability. On the other hand, the *structural condition focus strategy* $S_{5.3}$ leads to superior service life results and also the traffic performance as well as the visual performance are very satisfying—however, this is bought in terms of economic efficiency, a field in which this strategy only delivers medium range results.

A simple methodology for dealing with such individual strength and weaknesses is to identify and eliminate strategies that are performing worse in all aspects, when compared to other strategies. Such a simple strategy analysis starts from the assumption that a strategy is *dominated* by another *dominating* strategy when this *dominating* strategy leads to an at least equal performance with respect to all four indicators ($RT_{UA}$, $T_{SL}$, $A_{av}$, $C_{0,owner}$) and at the same time excels the *dominated* strategy in terms of at least one of these indicators.

In the case of the *Köhlbrandbrücke*, a clear domination by other strategies—in the sense that domination occurs for each and every single sample element—only applies for the reactive strategy $S_1$, which is clearly dominated by the preventive-proactive strategy $S_4$. As can be seen from Fig. 5.60, the strategy $S_4$ results in higher service life ($T_{SL}$) and traffic availability ($A_{av}$) as well as lower costs ($C_{0,owner}$) and time of unsatisfactory appearance ($RT_{UA}$) for each one of the 100 investigated sample elements, when compared to strategy $S_1$.

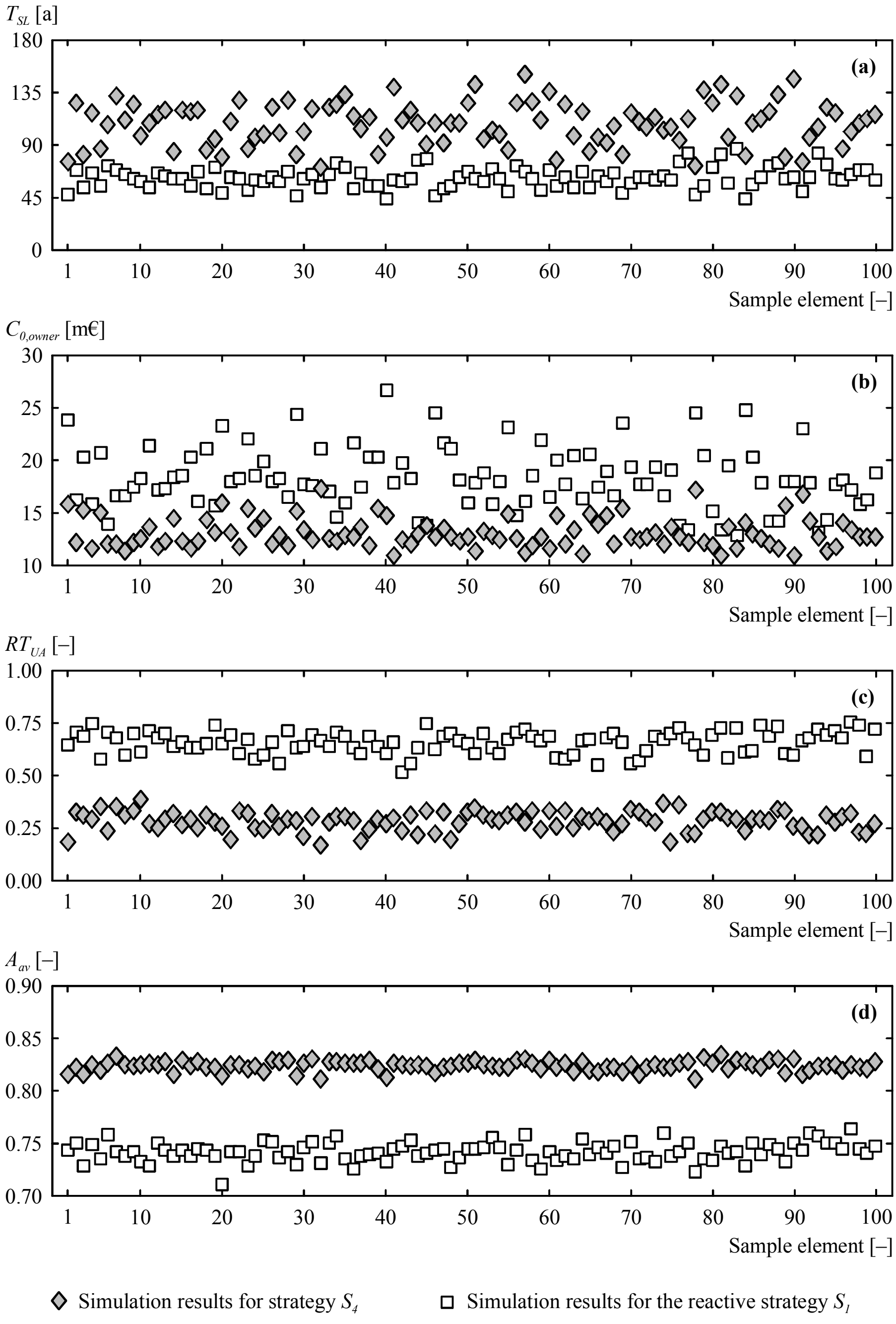



**Fig. 5.60:** Comparison of the maintenance strategies $S_4$ and $S_1$ in terms of dominance (Köhlbrandbrücke)

A less clear domination, which is based on the sample mean values of the simulation results for the performance indicators $T_{SL}$, $C_{0,owner}$, $RT_{UA}$, and $A_{av}$ (see Table 5-14) can be found for the following strategies:

- The *really applied strategy $S_0$* is dominated by the *proactive-preventive strategy $S_4$* and by the *structural-focus* strategy $S_{5.3}$.
- The *reactive strategy $S_1$* is dominated by all strategies, with the only exception of the *traffic-focus strategy $S_{5.1}$*, which yields worse results in terms of economic performance.
- The *proactive repair strategy $S_2$* is dominated by the strategies $S_{5.2}$ and $S_{5.3}$.

However, it must be kept in mind that this domination holds only true for the overall sample mean values—in the case of single sample elements, the performance ranking may occur the other way round. This, for example, happens in the case of strategy $S_0$, which is dominated by the preventive-proactive strategy $S_4$ with respect to the sample mean values of Table 5-14. Nonetheless, the seemingly inferior strategy $S_0$ still succeeds in outperforming the strategy $S_4$ in terms of costs in 31 of the 100 cases and in terms of traffic availability in 2% of the investigated cases, as it appears from Fig. 5.61. Similarly, the strategy $S_{5.3}$ is economically outperformed by the generally inferior strategy $S_2$ in 23% of the investigated cases (Fig. 5.62).

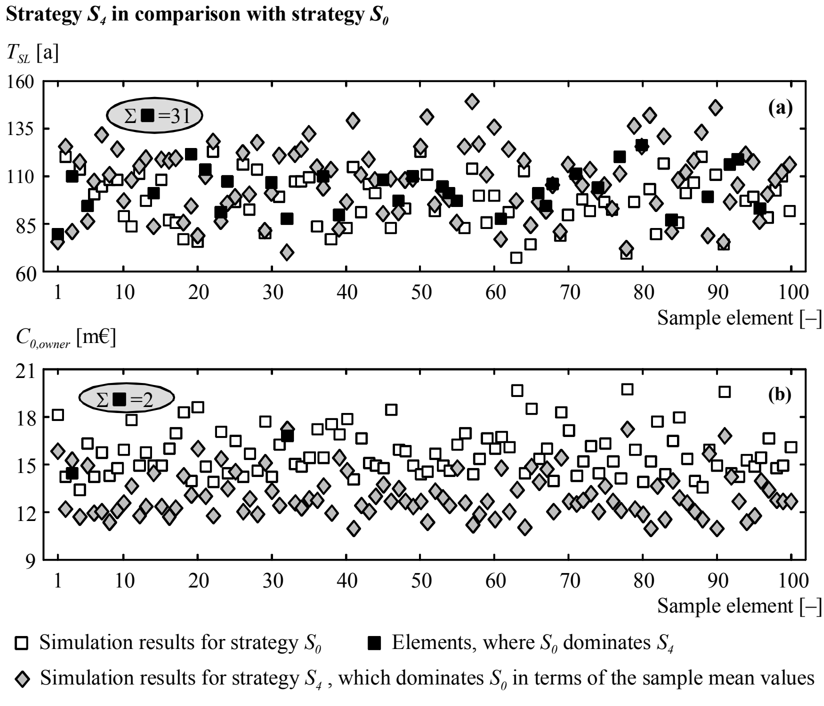



**Fig. 5.61:** Comparison of Maintenance Strategies $S_4$ and $S_0$ in terms of Dominance (Köhlbrandbrücke)

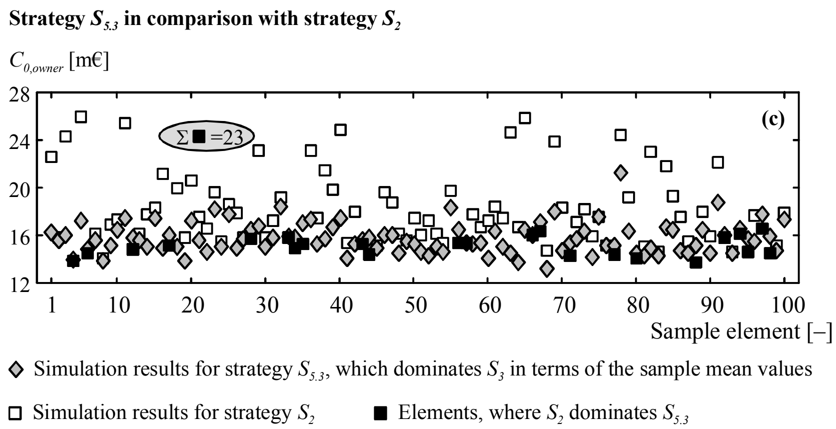



**Fig. 5.62:** Comparison of Maintenance Strategies $S_{5.3}$ and $S_2$ in terms of Dominance (Köhlbrandbrücke)

Analogous findings can be made for the *Brücke Bahrenfelder Chaussee*, for which the following dominance relationships can be found on the basis of sample mean values (see Table 5-15). However, these relationships only hold true for the sample mean values; vice-versa dominance relationships have been found on the level of individual sample elements.

- The *really applied strategy $S_0$* is dominated by the *appearance-focus strategy $S_{5.2}$*.
- The *reactive strategy $S_1$* is dominated by the *proactive-preventive strategy $S_4$* as well as by the *appearance-focus structural strategy $S_{5.2}$*.
- The *proactive repair strategy $S_2$* is dominated by *strategy $S_{5.3}$*.
- The *preventive repair strategy $S_3$* is dominated by *strategy $S_{5.2}$*.

In conclusion, it appears that the dominance-criterion still leaves five strategies in the case of the *Köhlbrandbrücke* ($S_3$, $S_4$, $S_{5.1}$, $S_{5.2}$, and $S_{5.3}$) and four strategies ($S_4$, $S_{5.1}$, $S_{5.2}$, and $S_{5.3}$) in the case of the *Brücke Bahrenfelder Chaussee* as potential candidates for selection. Each one of these remaining strategies may perform better than other strategies with respect to certain performance-aspects, but at the same time does underperform with respect to other aspects. From these remaining strategies, however, it is not possible to pinpoint the best one on the basis of plain dominance-analysis. Thus, a more elaborate methodology must be applied for comparing and evaluating the maintenance strategies under consideration of their specific strength and weakness. Only on this basis the best fitting strategy for the individual bridge can be identified.

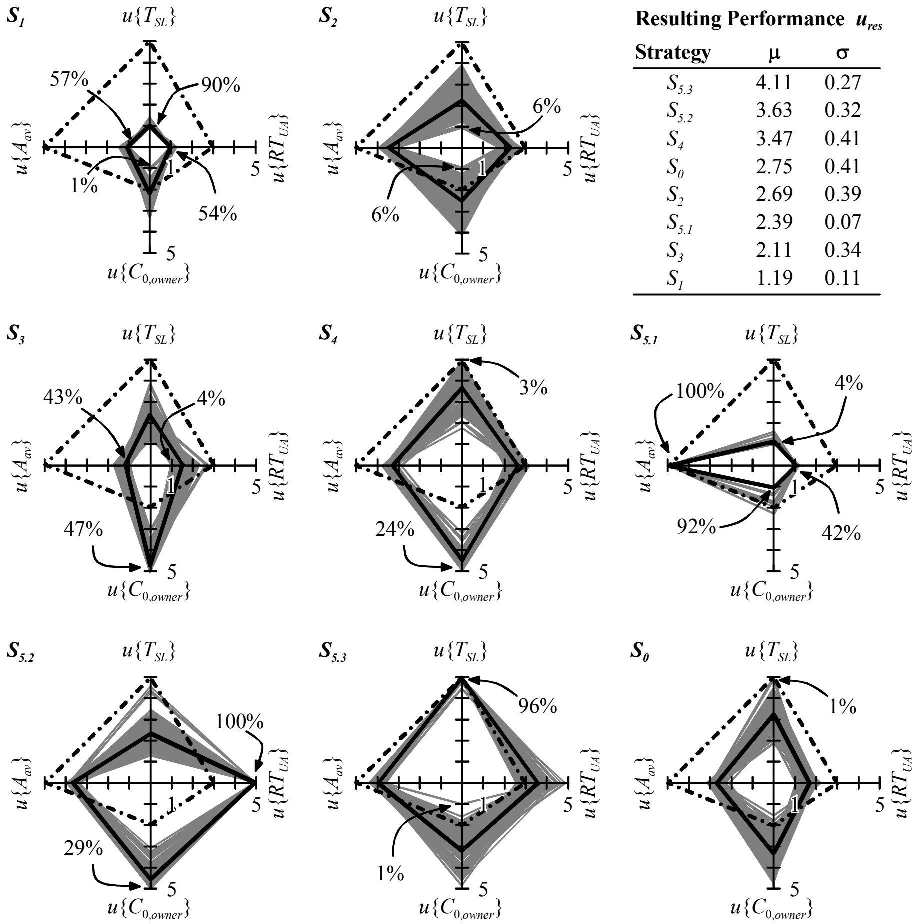
### 5.8.3 Strategy Evaluation with the Help of Performance Profile Analysis

**Köhlbrandbrücke**

The performance profiles for the *Köhlbrandbrücke*, as they are derived from the assessment results (see 5.7.10) according the procedure that is described in section 4.9, are depicted in Fig. 5.63. There it can be seen that the strategy $S_{5.3}$, which focuses on preserving a good structural condition, excellently agrees with the preference profile that has been identified for this bridge in sub-section 5.3 (see Fig. 5.3). This also coincides with providing the globally best resulting performance, which is expressed in terms of a sample mean value of $u_{res}$=4.11 (see Fig. 5.63, upper right). First of all, a long service life, which is a central requirement for this landmark bridge, is provided by this strategy: It leads to the longest service life of all considered strategies in 96% of the investigated sample elements; in terms of $T_{SL}$ it is only exceptionally exceeded thrice by strategy $S_4$ and once by strategy $S_0$. Also the traffic performance is superior to almost all strategies, as it can be anticipated from comparing the strategy profiles in Fig. 5.63. Further analysis of the simulation results shows that in terms of traffic availability, this strategy is only clearly dominated by the *traffic biased strategy* $S_{5.1}$ and partial dominance can be found only by strategy $S_{5.2}$, which leads to a superior traffic performance in just 12% of all investigated cases. Moreover, the visual as well as the economic performance both satisfy, or even exceed, the performance preferences which are found to be of medium importance ($u$=3 and 2 respectively). Besides, in terms of appearance this strategy ranks second after the consistently superior strategy $S_{5.2}$, and is only exceptionally dominated by strategy $S_4$ in just 2% of all cases, as has been found by further analysis. However, it should be noted that both the economic and visual performance results show a considerable scatter in the utility rating; as an effect it is even found that the strategy $S_{5.3}$ results in the worst economic performance of all strategies in one out of the 100 investigated cases.

The *appearance focus strategy* $S_{5.2}$ comes second in terms of the overall performance with a value of $u_{res}$=3.63 (Fig. 5.63). However, the general shape of the performance profile appears almost in the form of a mirror image of both the preference profile and the strategy profile of the above described strategy $S_{5.3}$: From the solid black line of the sample mean profile in Fig. 5.63, it appears that this strategy shows excellent results with respect to costs and appearance, while it only ranks third in terms of availability and only results in a medium service life. Consequently, this strategy should be discarded as a potential solution. It turns out that—against the seemingly good resulting performance $u_{res}$—this strategy simply has the wrong focus.

The *preventive- proactive strategy* $S_4$, the *real strategy* $S_0$, and the *preventive repair strategy* $S_3$ each show a similar profile with strength of service life and economic results and comparative weakness with respect to availability and appearance. Each one of the four strategies exhibits a considerable scatter in the service life results. In turn, the scatter in cost-results is found to be high for strategy $S_0$, while for strategy $S_4$ it is not—the latter one even economically outperforms all the other strategies in 24% of the investigated cases. The lowest overall rating of these four strategies is found for the *preventive repair strategy $S_3$*, which admittedly leads to a superior economic performance in 47% of the cases, but poorly performs in terms traffic performance.




**Resulting Performance $u_{res}$**

| Strategy | μ | σ |
|---|---|---|
| $S_{5.3}$ | 4.11 | 0.27 |
| $S_{5.2}$ | 3.63 | 0.32 |
| $S_4$ | 3.47 | 0.41 |
| $S_0$ | 2.75 | 0.41 |
| $S_2$ | 2.69 | 0.39 |
| $S_{5.1}$ | 2.39 | 0.07 |
| $S_3$ | 2.11 | 0.34 |
| $S_1$ | 1.19 | 0.11 |

Strategy-profiles of the individual sample elements ($N$=100)

Strategy-profile for the sample mean values of $u\{T_{SL}\}$, $u\{C_{0,owner}\}$, $u\{A_{av}\}$, and $u\{RT_{UA}\}$

Preference profile

...% Indication of how many of the extreme sample-element results (worst/best) are provided by each strategy: e.g. strategy $S_3$ excels all other strategies in terms of economic performance ($C_{0,owner}$) in 47% of the $N$=100 investigated cases, while it leads to the globally worst results with respect to traffic availability and aesthetic performance in 43% and 4% of the cases.

**Fig. 5.63:** Strategy Performance Profiles for the Köhlbrandbrücke compared to the preference profile (dash-dot line) that has been identified for the Köhlbrand-brücke in sub-section 5.3.

The *proactive repair strategy* $S_2$ reveals stable traffic performance and appearance results at intermediate levels and highly volatile results with respect to service life and costs. Generally, the low service-life results and the relatively low $u_{res}$-value disqualify this strategy for further consideration.

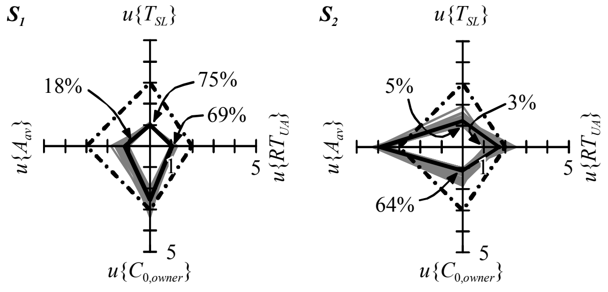
The poorest results are delivered by the traffic focus strategy $S_{5.1}$ and the reactive repair strategy $S_1$. While the first naturally excels all other strategies in terms of availability, it clearly underperforms in terms of all the other performance aspects: It leads to the second worst performance in terms of service life—just being slightly ahead of strategy $S_1$—and it also leads to the absolutely worst economic and aesthetic performance in 92% and 42% of the cases, respectively. The other poorly performing strategy, the *reactive strategy* $S_1$, clearly shows a performance profile that is inferior to all the other investigated strategies. In fact it is found that this strategy leads to the absolutely lowest service life in 90% of the sample elements. Also, it leads to the absolutely lowest performance with respect to traffic availability and appearance in 57% and 54% of the sample elements. Also the economic results for this strategy are consistently very poor. All in all, with a sample mean value for $u_{res}$=1.19 the *reactive strategy* $S_1$ leads to the lowest overall performance.

**Brücke Bahrenfelder Chaussee**

In the case of the *Brücke Bahrenfelder Chaussee*, the *preventive repair strategy* $S_4$ appears to be the one whose strategy performance profiles best coincides with the preference profile. According to the preference profile that has been identified for this bridge in sub-section 5.3 (see Fig. 5.4), the three aspects *service life*, *availability* and *costs* are equally important while the *visual appearance* is comparatively less important. As can be seen in Fig. 5.64, this relative weighting of the performance aspects is generally also found for the *preventive-proactive strategy* $S_4$; only the service life results seem to be merely at a modest level. However, the seemingly modest service life must be related to the results for the other investigated strategies. While strategy $S_4$ is clearly dominated by strategy $S_{5.3}$, of the remaining strategies only the strategy $S_{5.2}$ leads to better $T_{SL}$-results in 61% of the investigated sample elements, as has been found by further data analysis. Moreover, compared to the strategy $S_{5.2}$ the *preventive proactive strategy* $S_4$ shows very stable results with moderate scatter. This is one reason why the strategy $S_4$ also seems to be more suitable for the bridge Bahrenfelder Chaussee than the two strategies $S_{5.3}$ and $S_{5.2}$, both of which score higher in terms of the resulting performance indicator $u_{res}$. There are even further reasons for a comparative advantage of the preventive-proactive strategy $S_4$: The *structural focus strategy* $S_{5.3}$, on the one hand, leads to results in terms of visual appearance and traffic availability that are comparable to strategy $S_4$—however, the economic performance is considerably lower and also is characterised by higher scatter, while the service life performance is clearly dominating all other performance aspects. In other words, strategy $S_{5.3}$ buys its excellent service life results by higher maintenance costs. The appearance focus strategy $S_{5.2}$, on the other hand, leads to considerably scattering—and thus hardly projectable—service life results in combination with better economic results. In conclusion, the strategy $S_4$ seems to be more adept than these two strategies because it provides a more stable and—crucially—a more balanced performance in terms of service life and costs. The higher $u_{res}$-value of the other two strategies is rather caused by an overcompensation of service life by better economic and aesthetic performance in the case of strategy $S_{5.2}$ and overcompensation of higher costs by service life in the case of strategy $S_{5.3}$.

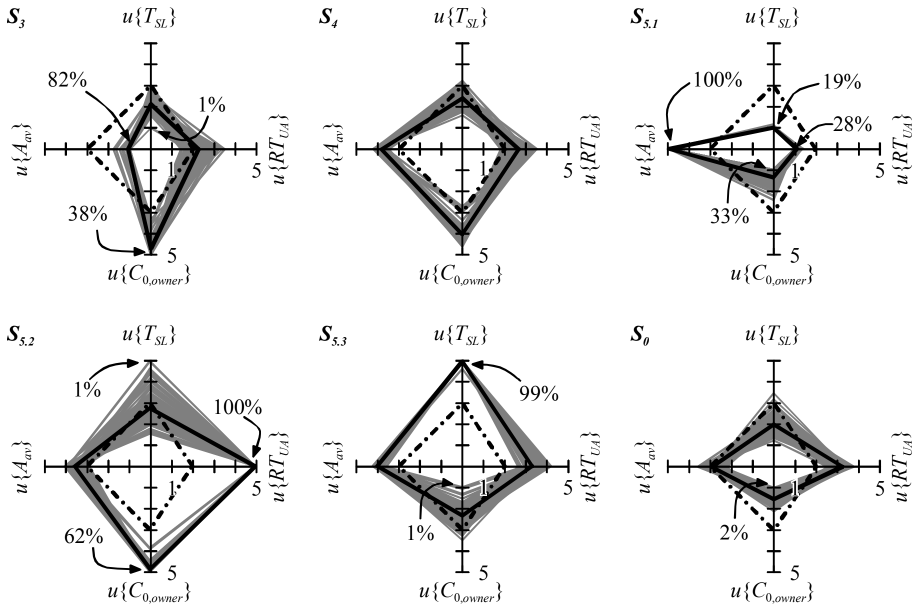
The really applied strategy $S_0$, in turn, is dominated by strategy $S_{5.2}$ as was found by the above described dominance analysis and thus can be neglected as a potential strategy. The preventive repair strategy $S_3$ clearly violates the traffic performance requirements of the Brücke Bahrenfelder Chaussee as it leads to the worst results of all strategies in 82% of the investigated cases.

The strategies $S_1$, $S_2$, and $S_{5.1}$ altogether obviously show very low performance results so that they can be left out of consideration as a potential strategy.




**Resulting Performance $u_{res}$**

| Strategy | μ | σ |
|---|---|---|
| $S_{5.2}$ | 3.97 | 0.34 |
| $S_{5.3}$ | 3.69 | 0.21 |
| $S_4$ | 3.26 | 0.32 |
| $S_3$ | 2.54 | 0.32 |
| $S_{5.1}$ | 2.21 | 0.12 |
| $S_2$ | 2.00 | 0.20 |
| $S_0$ | 2.36 | 0.30 |
| $S_1$ | 1.47 | 0.16 |




Strategy-profiles of the individual sample elements ($N$=100)

Strategy-profile for the sample mean values of $u\{T_{SL}\}$, $u\{C_{0,owner}\}$, $u\{A_{av}\}$, and $u\{RT_{UA}\}$

Preference profile

...% Indication of how many of the extreme sample-element results (worst/best) are provided by each strategy: e.g. strategy $S_4$ excels all other strategies in terms of $T_{SL}$ in 3% of the $N$=100 investigated cases, while in 24% of the 100 cases it leads to the highest costs

**Fig. 5.64:** Strategy Performance Profiles for the Brücke Bahrenfelder Chaussee (solid line) compared to the maintenance objectives preference profile (dashed line) that has been identified for the Bahrenfelder Chaussee in sub-section 5.3.

### Detailed Discussion of the Results for the Really Applied Strategy $S_0$

For both bridges a closer look at the resulting performance profiles of the really applied strategy $S_0$ reveals noticeable discrepancies to the bridge specific preference profiles and to the results of the well performing strategies. When looking at the results for the *Köhlbrandbrücke* in Fig. 5.63, it appears that the traffic performance ($A_{av}$), which is considered to be of very high importance for this bridge, is clearly underrepresented. Likewise the expected service life results ($T_{SL}$), a major point of interest for the *Köhlbrandbrücke*, also look somewhat underrepresented. As opposed to this, the moderate economic performance ($C_{0,owner}$) appears still better than necessary for this bridge, for which maintenance costs are of lesser priority and thus trading off economic performance for both an increased service life and traffic availability would be a rational choice.

In the case of the *Brücke Bahrenfelder Chaussee,* the preference profile for this bridge puts the same weight on service life, availability and economic performance while it puts less emphasis on the visual appearance. In contrast to this, the resulting performance profile of the really applied strategy $S_0$ shows strikingly weak service life and economic performance in favour of better results in terms of availability and visual appearance (Fig. 5.64).

For drawing conclusions from these discrepancies, it is crucial to keep in mind that the performance profiles from their very definition of the *u*-values by equation 4.70 only put one strategy into perspective of alternative strategies. For instance, the performance profile values of $u\{T_{SL}\}$=3 and $u\{\mathrm{A}_{av}\}$=2.5, as they approximately can be found as sample mean values for the Köhlbrandbrücke (Fig. 5.63), only reveal that in average the service life $T_{SL}$ for the $S_0$-strategy lies halfway between the best results and the worst results obtained by all investigate strategies while the traffic performance $\mathrm{A}_{av}$, in average, lies at 3/8 on the way from the worst to the best results of all strategies. Consequently, the performance profile results do not indicate how good or bad a strategy is *per se*. Rather, they are helpful for comparing the *really applied strategy $S_0$* to alternative—and it must be emphasized: *hypothetical*—strategies by indicating comparative strengths and weaknesses.

Thus, it can be concluded from the results in Fig. 5.63 and Fig. 5.64 that the SAMS-program proves to be a helpful tool for indicating maintenance strategies, which—under consideration of the limitations from using a theoretical model—could in the long run lead to a better performance than the really applied strategies. Specifically, the results indicate that the performance results of really applied $S_0$-strategy in the case of the *Köhlbrandbrücke* show comparative weakness in terms of traffic availability and service life, which could be overcome by introducing elements of the theoretically better performing strategy $S_{5.3}$ into the maintenance application rules. In the case of the *Brücke Bahrenfelder Chaussee,* the simulation results indicate that the comparatively weak service life results as well as the low economic performance could be enhanced by making strategy $S_0$ look more like strategy $S_4$.

However, it must be kept in mind that these implications only hold true when the alternative strategy is being applied from the beginning and over an infinite time horizon. Changing strategies at some time during service life actually creates a completely new strategy which must be assessed and evaluated over the whole time horizon. This can be easily understood from the example of changing lately from a reactive strategy to a preventive strategy, which may not be effective because of the already existing pre-damage. Moreover, such a change of the applied strategy may even lead to increased costs due the additional impregnation or coating works.

**Inter-Bridge Comparison of the Performance Profiles**

A comparative examination of the performance profiles in Fig. 5.63 and Fig. 5.64 shows that the performance patterns of the strategies are mostly consistent for both bridges in such a way that the majority of strategies exhibit a similar shape of the performance profile, irrespective of the bridge they are applied to. However, an exception from this is found for the two proactive strategies $S_2$ and $S_4$, which result in a lower performance with respect to service life ($T_{SL}$) and maintenance costs and ($C_{0,owner}$) in the case of the *Brücke Bahrenfelder Chaussee* so that the profile shape appears significantly flatter compared to that of the *Köhlbrandbrücke*.

The significant differences in the results can be explained by the structural differences between the two bridges, which lead to the clearly diverging failure modes that are described above in sub-section 5.7.11 with reference to Fig. 5.46 and Fig. 5.50. A closer analysis of the simulation results shows that the prevailing shear failure at intermediate support '01V' of the *Brücke Bahrenfelder Chaussee* cannot be significantly delayed by the proactive intervention regime of $S_2$ and $S_4$. The reason for this is that proactive shear reinforcement conservation at '01V' only exceptionally occurs because the shear capacity at the other locations '02' and '0.5' is usually still far from critical before the ultimate failure at '01V' (Fig. 5.52 (f) and (i)). This is why the failure mode distribution is basically unchanged between $S_1$ and $S_2$ (Fig. 5.50). The same is true for strategy $S_4$, where the extended service life (compared to strategy $S_1$ and $S_2$) mainly can be ascribed to the preventive impregnation works, and thus no advantage is given over strategy $S_3$—as can be concluded by comparing the results for the simply preventive strategy $S_3$ (Fig. 5.52 (a) and (d)) with the proactive-preventive strategy $S_4$ (Fig. 5.53 (c) and (f)). Apparently, the situation is completely different for the *Köhlbrandbrücke* where proactive tendon conservation at '01.5' comes into effect and decisively contributes to a longer service life compared to strategy $S_3$ (Fig. 5.48 (a) and (d), Fig. 5.49 (b) and (e)).

The differences in the maintenance costs $C_{0,owner}$ can be simply traced back to the above described implications on the service life. Due to the lower discounting effect of a shorter service life, also the economic performance does not benefit from the application rules of strategies $S_2$ and $S_4$ in the case of the *Bahrenfelder* bridge.

### 5.8.4 Conclusion

Three major findings can be made from the evaluation of eight different strategies for the two case study bridges.

The first finding is that the evaluation methods turned out to be helpful for isolating a strategy for each bridge, which of all the investigated strategies is most suitable for satisfying the bridge specific performance requirements. In the scope of the actual investigation, these are the *structural focus strategy $S_{5.3}$* in the case of the *Köhlbrandbrücke* and the *preventive-proactive strategy $S_4$* in the case of the *Brücke Bahrenfelder Chaussee*. However, it is also found that each one of the identified strategies still exhibit certain weaknesses. In the case of the Köhlbrandbrücke, the *strategy $S_{5.3}$,* which focuses on preserving a good structural condition, is found to be the most appropriate strategy in view of the preference profile of the *Köhlbrandbrücke*. However, the traffic availability performance is slightly underrepresented by this strategy. In the case of the *Brücke Bahrenfelder Chaussee*, on the other hand, the *preventive-proactive strategy $S_4$* turned out to be most suitable in terms of the resulting performance profile. Nevertheless, the resulting service life still is slightly underrepresented when compared to both the economic and the traffic performance. Accordingly, both strategies could be further improved: The strategy $S_{5.3}$ in the case of the *Köhlbrandbrücke* benefits from additional or adjusted maintenance rules that provide for better traffic performance and the strategy $S_4$ in the case of the *Brücke Bahrenfelder Chaussee* can be enhanced by introducing additional service life extending measures.

Secondly, it can be concluded from the simulation results that the SAMS-program is a helpful tool for indicating maintenance strategies, which—under consideration of the limitations from using a theoretical model and—can lead to a better long-term performance than the really applied strategies.

The third finding is that the resulting performance patterns of the investigated strategies are mostly consistent for both investigated bridges. However, there is also evidence that structural peculiarities of the bridges can significantly influence the resulting performance profile and thus can lead to diverging performance patterns in inter-bridge comparison. This latter observation strengthens the argument that maintenance strategies must always be assessed and evaluated in the bridge specific context.

## 5.9 Strategy Improvement

### 5.9.1 Overview

The previous section indicates that the prominent strategies still do have some shortcomings with respect to certain performance aspects. Therefore, it seems promising to use these strategies as a starting point for developing improved strategies. This can be done by heuristically adapting the strategies' underlying maintenance application rules in order to heal their shortcomings. In easy cases, this can be simply achieved by tightening selected condition thresholds for maintenance application, as for example earlier pavement resurfacing for improving the average traffic availability. However, it must be kept in mind that strategy improvement with respect to one performance aspect, for example more frequent concrete repair, can lead to worsened performance with respect to other aspects, such availability and maintenance costs. Also, economic improvement—even as an isolated objective—is usually less obviously to achieve than the above described traffic availability improvement by intensified pavement maintenance. In such less obvious cases more elaborate heuristic improvement can be based on combining characteristics of good performing strategies—such as condition thresholds and combination rules—in order to find superior strategies. In these cases, heuristic strategy improvement usually necessitates iterative strategy adaptation, performance assessment, and evaluation for finding a better strategy.

In the following, a heuristic strategy improvement will be conducted for the two case study bridges. As described above, the improved strategies are developed by using the decision rules of the best-performing strategy as a starting point and by adapting decision criteria referring to the performance aspects that show comparative weakness. Then, the SAMS program is used for developing strategies that are tailored to meet the bridge specific performance requirements to an even higher degree.

### 5.9.2 Köhlbrandbrücke

As it is found, the *structural condition focus strategy* $S_{5.3}$ leads to the most appropriate performance profile of all strategies that have been investigated so far. Its only—if relatively small—weakness is found in terms of the traffic performance. Accordingly, the approach for further improvement of strategy $S_{5.3}$ simply consists in intensifying the proactive asphalt and expansion joint maintenance regime because the resulting availability '*a*' (equation 4.67) depends on the minimum value of both the availability from expansion joint and pavement condition. On the one hand, this is done by introducing an additional proactive threshold with respect to the resulting pavement condition $CI_{A,res}$, whose violation triggers pavement maintenance along the entire bridge. On the other hand, an analogous expansion joint maintenance regime with cyclic alternating repair and replacement and a $CI_{Exp,res}$-threshold is introduced. Such intensified intervention rules considerably strengthen the proactive nature of strategy $S_{5.3}$ which only implements selective pavement maintenance to the worst deteriorated bridge ramp or joint when a condition $CI_{A,min}$=0.375 or $CI_{Exp,res}$=0.5 is reached (see sub-section 5.6.2).

In order to do so, two potential strategies are defined. The first one, here referred to as strategy $S_{Imp,1}$, modifies the strategy $S_{5.3}$ only in terms of a stricter pavement maintenance regime by introducing an additional performance threshold $CI_{A,res}$ =0.5 for pavement resurfacing—or replacement, when two resurfacings have been previously applied—along the entire bridge. By doing so, pavement condition will move within a narrower bandwidth from 0.5 to 1.0 compared to strategy $S_{5.3}$ (see Fig. 5.42 (d) and (e)).

The second strategy $S_{Imp,2}$ investigates a further improvement from securing an overall expansion joint condition above $CI_{Exp,res}$ =0.625. By this means the adverse effects of the higher volatility of the expansion joint condition (Fig. 5.42 (e)), compared to the pavement condition, as it occurs for strategy $S_{5.3}$ (Fig. 5.42 (d)), can be reduced.

The resulting performance profiles for the improved strategies $S_{Imp,1}$ and $S_{Imp,2}$ are depicted in Fig. 5.65, where they are compared to those of the *structural focus strategy* $S_{5.3}$. There it can be found that the improved strategy $S_{Imp,1}$ slightly improves the traffic performance without significantly compromising the other performance aspects**.** As can be seen in Table 5-16, the sample mean values for the service life $T_{SL}$ and the aesthetic performance $RT_{UA}$ do hardly differ between the improved strategy $S_{Imp,1}$ and the strategy $S_{5.3}$, while the traffic performance $A_{av}$ and the maintenance costs $C_{0,owner}$ are slightly enhanced by around 1%. The latter decrease in service life costs can simply be explained from the fact that the relatively cheap pavement resurfacing works of strategy $S_{Imp,1}$ postpone the more expensive pavement replacement works and thus beneficial discounting effects apply.

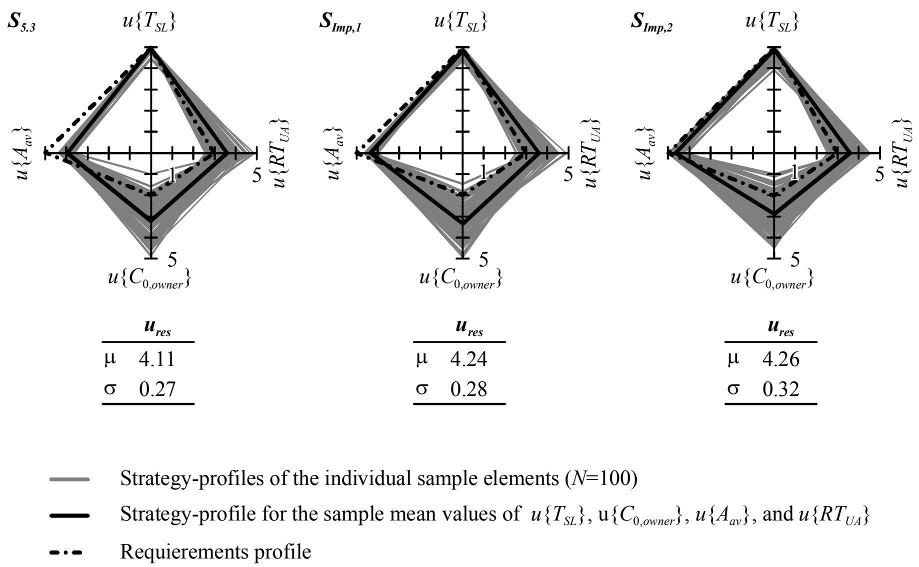



**Fig. 5.65:** Strategy performance profiles of the Köhlbrandbrücke for the strategy $S_{5.3}$ and the two improved strategies $S_{Imp,1}$ and $S_{Imp,2}$.

The alternative strategy $S_{Imp,2}$, with its intensified expansion joint maintenance regime, leads to an even further increased traffic performance. However, as it appears from Table 5-16, this increase in *average traffic availability* $A_{av}$ occurs only in the range of 1% and is bought by disproportionally higher service life costs, which increase by 5% from 15.7 m€ to 16.5 m€. The decrease in service life by around one year, which at first sight seems strange because an even more proactive expansion joint maintenance should lead to reduced shear reinforcement corrosion and thus to an extended service life, can be explained as follows: The effects of the highly proactive shear reinforcement conservation regime, which applies conservation works along the entire bridge independent from the actual critical location, are reduced because slower deterioration at the individual SAMS elements also lead to a reduced number over overall proactive conservation works. As a consequence, conservation works at the critical shear elements '117' and '101' (see Fig. 5.46) are carried out less frequently for strategy $S_{Imp,2}$ because of delayed limit state violations at the other elements.

**Table 5-16:** Parameters for the Performance Indicator Distributions (Köhlbrand-brücke)

| | $T_{SL}$ [a] | | $C_{0,owner}$ [$10^3$ €] | | $RT_{UA}$ [–] | | $A_{av}$ [–] | |
|---|---|---|---|---|---|---|---|---|
| **Strat.** | μ [a] | σ | μ [a] | σ | μ [a] | σ | μ [a] | σ |
| $S_{5.3}$ | 131 ± 4 | 20 | 15.7 ± .3 | 1.3 | .44 ± .012 | .06 | .85 ± .001 | .004 |
| $S_{Imp,1}$ | 131 ± 4 | 20 | 15.6 ± .3 | 1.3 | .44 ± .011 | .06 | .86 ± .001 | .005 |
| $S_{Imp,2}$ | 130 ± 4 | 19 | 16.5 ± .3 | 1.3 | .44 ± .011 | .06 | .87 ± .001 | .005 |

[a] The sample mean value is expressed in terms of its 95% confidence interval.

These effects also appear from a dominance analysis, whose results are represented in Table 5-17: Compared to the strategy $S_{5.3}$, the improved strategy $S_{Imp,1}$ is found to cause a superior traffic performance in 100% of the investigated cases as well as lower service life costs in 77 of 100 cases, while regarding service life and appearance the picture appears rather balanced with ratios of 48:52 and 43:57. In the case of the strategy $S_{Imp,2}$, the Table 5-17 highlights the trade-off between increased traffic availability and maintenance costs: $S_{Imp,2}$ excells $S_{Imp,1}$ in 99% of the cases in terms of traffic availability—however, it also causes higher costs in 99% (=100% - 1 %) of the observed cases.

**Table 5-17:** Dominance Analysis (Köhlbrandbrücke)

| **Strategy Comparison** [a] | $T_{SL}$ **[%]** | $C_{0,owner}$ **[%]** | $RT_{UA}$ **[%]** | $A_{av}$ **[%]** |
|---|---|---|---|---|
| $S_{Imp,1} > S_{5.3}$ | 48 | 77 | 43 | 100 |
| $S_{Imp,2} > S_{5.3}$ | 45 | 1 | 43 | 100 |
| $S_{Imp,2} > S_{Imp,1}$ | 53 | 1 | 52 | 99 |

[a] The comparison ">" refers to the qualitative utility of each performance aspect, which means that in the case of $T_{SL}$ and $A_{av}$ ">" applies to higher numerical values, whereas in the case of $C_{0,owner}$ and $RT_{UA}$ ">" refers to lesser numerical values.

In conclusion, the strategy $S_{Imp,1}$ is suitable for obtaining a service life performance that satisfies all requirements according to their bridge specific relevance. The resulting strategy performance profiles not only show a very close match with the preference profile, but it also excels the previously best performing strategy $S_{5.3}$ by an increased traffic performance which is accompanied by reduced maintenance costs. As opposed to this, the strategy $S_{Imp,2}$ seems not to be suitable because of the expensively bought traffic performance increase and the slightly reduced service life. However, under consideration of the relative weighting of traffic availability and maintenance costs within the preference profile (5:2) even the found relation of +1% $A_{av}$ to + 5% $C_{0,owner}$ still may be worth a second thought.

### 5.9.3 Bahrenfelder Chaussee

In the case of the *Brücke Bahrenfelder Chaussee*, strategy improvement aims at further increasing the performance results of strategy $S_4$. As it can be seen in the above Fig. 5.50, the service life for this bridge is mainly determined by shear failure at the intermediate support and, to a lesser extent, by bending failure in the eastern span. Consequently, heuristic strategy improvement is achieved by implementing maintenance application rules that delay shear and bending failure at the respective locations. In order to do so, two potential strategies $S_{Imp,1}$ and $S_{Imp,2}$ are defined.

The first one, here referred to as strategy $S_{Imp,1}$, modifies the *preventive-proactive strategy* $S_4$ in such a way that shear reinforcement conservation at axis 01 as well as tendon conservation at the low points of the eastern span are proactively applied whenever concrete girder repair becomes necessary. The second strategy variant, which is referred to as strategy $S_{Imp,2}$, also applies this proactive reinforcement conservation, but in addition uses a stricter application threshold for girder concrete repair of $CI_{CS,res}$=0.50 compared to 0.375 as it is applied for strategy $S_4$.

The resulting performance profiles for the two improved strategy variants appear in Fig. 5.66, where they are compared to those of the *preventive-proactive strategy* $S_4$. There it can be found in Fig. 5.66 (b) that the improved strategy $S_{Imp,1}$ almost perfectly matches the bridge specific preference profile in the sense that the aspects $T_{SL}$, $C_{0,owner}$, and $A_{av}$ are equally well represented while the aesthetic indicator $RT_{UA}$ is allowed to be remain on a comparatively lower level. In absolute terms, the improved strategy $S_{Imp,1}$ leads to a substantial increase in service life, as it appears in the form of sample mean value of 156 years compared to 136 years, which is accompanied by only marginal changes in the other performance results, as can be seen in Table 5-18.

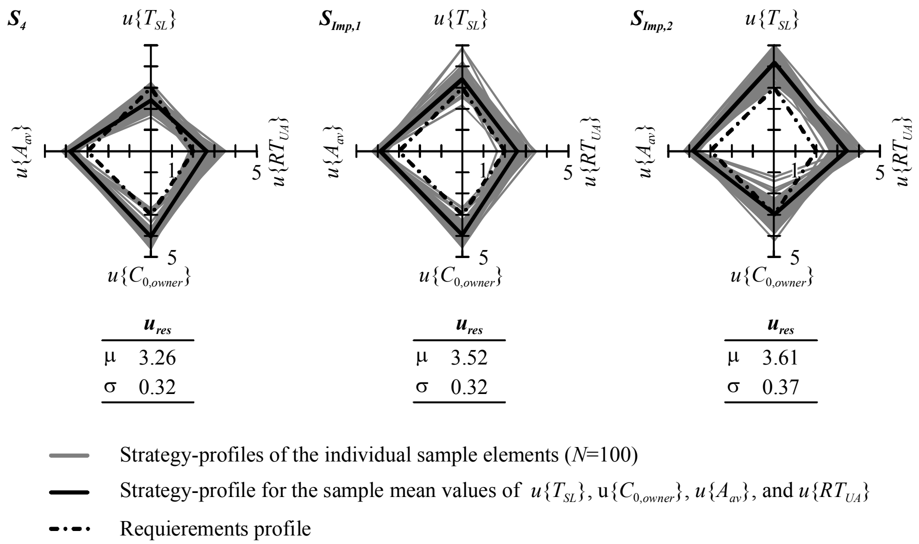



**Fig. 5.66:** Strategy performance profiles of the *Brücke Bahrenfelder Chaussee* for the strategy $S_4$ and the two improved strategies $S_{Imp,1}$ and $S_{Imp,2}$.

In contrast to this, the performance profile of the alternative strategy $S_{Imp,2}$ does not lead to an enhanced match with the preference profile. In fact, the profile rather appears in the form of an upside-down image of the $S_4$-strategy-profile, it was originally supposed to improve (Fig. 5.66 (c)). Specifically, both Fig. 5.66 and Table 5-18 show that this strategy leads, as it was intended, to a considerably increased service life (μ=172±5 years). However, this is bought at the expense of noticeably higher costs with a mean increase by around 16% compared to the $S_4$-results—with increases ranging from 7.5% to up to 31.7% for individual sample elements, as can be seen from more detailed investigations. In addition, the aesthetic performance is considerably enhanced by around 13%, which however is not of major interest for this bridge in view of the preference profile.

**Table 5-18:** Parameters for the Performance Indicator Distributions (Brücke Bahrenfelder Chaussee)

| | $T_{SL}$ [a] | | $C_{0,owner}$ [$10^3$ €] | | $RT_{UA}$ [–] | | $A_{av}$ [–] | |
|---|---|---|---|---|---|---|---|---|
| **Strat.** | μ [a] | σ | μ [a] | σ | μ [a] | σ | μ [a] | σ |
| $S_4$ | 134 ± 4 | 19 | 424 ± 8 | 42 | .510 ± .006 | .03 | .879 ± .001 | .007 |
| $S_{Imp,1}$ | 154 ± 5 | 24 | 429 ± 8 | 42 | .514 ± .006 | .03 | .881 ± .001 | .007 |
| $S_{Imp,2}$ | 169 ± 5 | 28 | 494 ± 11 | 54 | .443 ± .005 | .03 | .880 ± .001 | .007 |

[a] The sample mean value is expressed in terms of its 95% confidence interval.

A closer look to the results of individual sample elements in the form of a dominance analysis confirms these findings. As can be seen in Table 5-19, the improved strategy $S_{Imp,1}$ consistently leads to a longer service life compared to the *preventive-proactive strategy $S_4$* in 100% of the sample. Moreover, it is found that the increase in service life is substantial and reaches improvements from a minimum comparative improvement by 2% up to 35% for individual sample elements and a sample mean increase by around 15%, as can be found by detailed analysis. With respect to the other performance indicators $C_{0,owner}$, $RT_{UA}$, and $A_{av}$ no consistent dominance relationship can be found. First, the traffic performance $A_{av}$ may seem to be superior in the case of strategy $S_{Imp,1}$ for a clear 81%-majority of sample elements; however, a detailed analysis shows that the performance differences in absolute values are consistently below 1% and thus are considered to be too small for making a difference. Second, the aesthetic results are better for the strategy $S_{Imp,1}$ compared to $S_4$ almost an equal number of cases than the reverse, namely in 44 of the 100 sample elements. This can be easily explained by the fact that the intensified proactive tendon and shear reinforcement conservation is not—at least in the scope of the actual investigation—considered to affect the visual appearance; thus different outcomes for the $RT_{UA}$-values solely depend on the remaining service life after the last concrete repair intervention for each sample element. Thirdly, the differences in the economic performance, similarly to those of the traffic performance, are ranging from +3.6% to -5.3%.

The alternatively investigated strategy $S_{Imp,2}$ also clearly dominates the *preventive-proactive strategy $S_4$* in 100% of the sample in terms of the $T_{SL}$-results. Moreover, it results in such high service life results that it also clearly dominates the other improved strategy $S_{Imp,1}$, which shows better $T_{SL}$-values in only 3% of the cases. Nevertheless, the strategy $S_{Imp,2}$ shows a consistently poorer economic performance than the strategies $S_{Imp,1}$ and $S_4$, by which it is dominated in 100% of the cases. Conversely, the strategy $S_{Imp,2}$ clearly dominates these two strategies in terms of the visual appearance; a fact that easily can be attributed to the proactive concrete repair works. Analogously to the above described strategy $S_{Imp,1}$, the value-differences in the traffic performance results are found to range from +0.6% to -0.4% and thus are too small to be relevant.

**Table 5-19:** Dominance Analysis (Brücke Bahrenfelder Chaussee)

| **Strategy Comparison** [a] | $T_{SL}$ **[%]** | $C_{0,owner}$ **[%]** | $RT_{UA}$ **[%]** | $A_{av}$ **[%]** |
|---|---|---|---|---|
| $S_{Imp,1} > S_4$ | 100 | 19[c] | 44 | 81 |
| $S_{Imp,2} > S_4$ | 100 | 0 | 100 | 64 |
| $S_{Imp,1} > S_{Imp,2}$ | 3 | 100 | 0 | 73 |

[a] The comparison “>” refers to the qualitative utility of each performance aspect, which means that in the case of $T_{SL}$ and $A_{av}$ “>” applies to higher numerical values, whereas in the case of $C_{0,owner}$ and $RT_{UA}$ “>” refers to lesser numerical values.

In conclusion, the strategy $S_{Imp,1}$ turns out to be very suitable for obtaining an overall performance of the *Brücke Bahrenfelder Chaussee* in accordance with the bridge specific requirements. Of all investigated strategies, it shows the closest match with the preference profile in terms of the relative performance levels regarding the aspects service life, costs, appearance and traffic flow. Also, the performance with respect to the aspects that are identified to be of importance for this bridge— service life, costs, and traffic flow—reaches high levels.

Because of this close match in general shape at high performance levels, the strategy $S_{Imp,1}$ is more appropriate than the alternatively investigated improved strategy $S_{Imp,2}$—even if that strategy results in a higher resulting performance with a sample mean value $u_{res}$=3.61 compared to $u_{res}$=3.52 (Fig. 5.66). This is because of the high $u_{res}$-value results from an overemphasis of service life which is bought by considerably increased maintenance costs. This is a good example for the reason given in sub-section 2.8.5 for the introduction of the performance-profile related strategy evaluation.

## 5.10 Contribution of the Approach

Critics may argue that the simulation results merely confirm the intended effect of a strategy and are therefore obvious. The *traffic focus* strategy $S_{5.1}$, for example, with its early asphalt replacement in combination with expansion joint maintenance apparently leads to high traffic availability, while the *appearance focus* strategy $S_{5.2}$ t unsurprisingly maintains the aesthetic appearance by a repeated application of concrete coating and early concrete repair.

However, such objections are only sustainable as long as the main strategy focus, such as traffic availability or appearance, is concerned. They lose ground when relating to other performance aspects. Predominantly the strategy-effects are usually less obvious, as for example the economic and aesthetic performance of the traffic focus strategy. Moreover, the whole performance-spectrum of less object-oriented strategies such as a '*reactive strategy*' or '*preventive-proactive strategy*' is far from being obvious and a detailed investigation becomes necessary. Also, the above discussed influence of the bridge specific context on the strategy performance results (see sub-section 5.8.3), which was only recognised by inter-bridge comparison of the results for the strategies $S_2$ and $S_4$ and a subsequent detailed analysis, proofs the usefulness of the approach.

Moreover, both the introductory discussion of the simple dominance analysis (see 5.8.2) and the later discussed strategy improvement results (see 5.9) show that usually no Pareto optimal strategies can be identified. This confirms the need to identify bridge specific strength and weaknesses of strategies and evaluate these in a differentiated way under consideration of the specific needs and requirements.

In summary, the newly developed approach delivers profound and valuable performance results on the basis of a sophisticated simulation procedure. It thoroughly takes into account the interdependencies between structure, deterioration and maintenance effects that occur in the bridge specific context. Thus, it allows for capturing influences that moderate or even decisively control the outcomes of maintenance strategies—especially influences that are hidden by the complexity of the system and otherwise may have been unaccounted for.

## 5.11 Conclusion

This chapter describes a comprehensive simulation study on maintenance strategy assessment and evaluation. It deals with the application of the SAMS-program for assessing and evaluating alternative maintenance strategies by using the example of two case-study bridges in Hamburg. The data for building the SAMS models are based on available documentation of the two bridges whenever possible; alternatively relevant literature or estimates are used. Altogether eight different maintenance strategies are investigated, of which only the really applied strategy differs between the two bridges in terms of the underlying maintenance application rules. The other seven hypothetical strategies represent a wide spectrum of maintenance application rules, ranging from a strictly reactive approach to a sophisticatedly tailored strategy that aims at minimising structural deficits. In summary, the investigation leads to the following major findings:

First, it can be concluded that each maintenance strategy has its individual strengths, where it excels other strategies; but at the same time each strategy also has characteristic shortcomings, where it falls back behind other strategies. Therefore no all-dominating (Pareto optimal) strategies could be found. It also appeared that in most cases the strategy performance profiles are consistent for the two bridges. However, there is also evidence that structural peculiarities of the bridges can significantly influence the resulting performance profile and thus can lead to inconsistent performance patterns in inter-bridge comparison.

Second, for the example of the two case study bridges it was found that specific strategies could be identified which excel other strategies because with a view to their performance profile they promise to be superior in helping the bridge to fulfil its specific requirements. Thus, the results indicate the practical applicability of the SAMS-program as a helpful tool for maintenance planning.

Third, an example application shows that the SAMS-program can successfully be used for developing maintenance strategies that are tailored to the bridge specific requirements. The demonstrated method for heuristic strategy improvement, by which the best-performing strategy from a preliminary investigation is used as a starting point for adaptation of the maintenance application rules, proved to be a promising approach.

# 6 Multi-Objective Bridge Maintenance – A New Perspective

## 6.1 Introduction

This chapter summarises the investigation at hand. Specifically, it makes clear the main achievements and their contribution to the actual state of research and bridge maintenance practice. As a final step, the chapter highlights impulses of the research results on practical bridge maintenance and outlines open issues for future investigations.

## 6.2 Research Needs and Starting Point of the Investigation

From an ideal point of view, bridges should be designed, constructed and operated in such a way that they fulfil requirements from the fields *function*, *economy*, *society & culture*, and *environment*. In theory, these requirements should be defined during a so-called *requirements development phase*—a central process within bridge management (Fig. 6.1). Furthermore, they constitute the basis for the development of design and maintenance objectives, which then guide the design and maintenance processes and constitute the basis for strategic maintenance planning (Fig. 6.1).

In essence, the strategic maintenance planning process comprises four steps. The process begins with the identification of the performance requirements and the definition of the maintenance objectives. The second step includes the definition and simulation of alternative strategies and the assessment of these strategies with the help of specific indicators, which quantitatively describe the individual performance aspects such as function or economy. The third step—strategy evaluation—is defined by the interpretation of the assessment results with respect to performance requirements as well as strategic maintenance objectives. Ultimately, the suitable maintenance strategy is identified and the long term maintenance plan is formulated. In order to deliver reliable recommendations for actual maintenance implementation, these planning procedures should—if possible—take into account bridge specific data as they are available from bridge inspections.

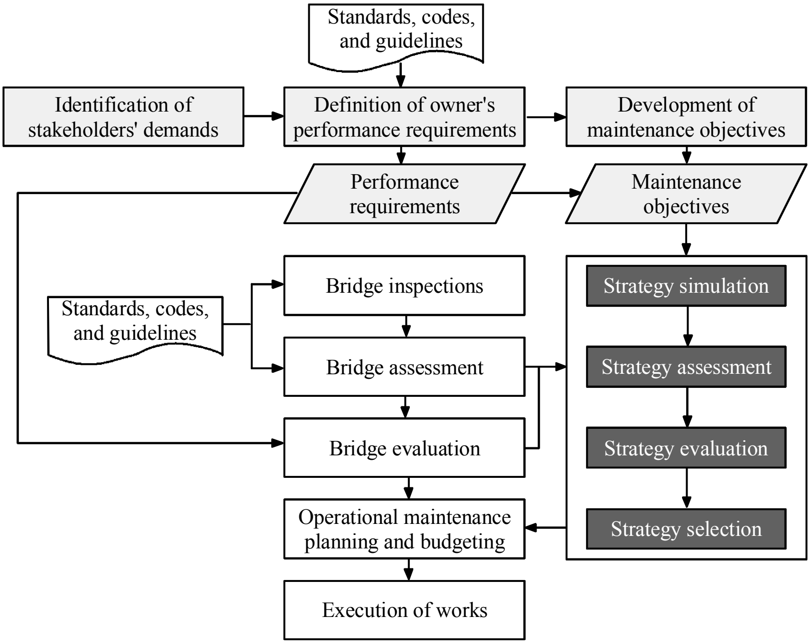



**Fig. 6.1:** Requirements and objectives development phase (light grey shaded) and maintenance planning process (dark shaded, white text colour) as well as the associated bridge management activities and input data.

However, an empiric evidence for the general need for bridge specific consideration of requirements—a necessary precondition for multi-aspect and bridge specific maintenance planning—is not found in the relevant literature. Also, recommendations for structured approaches to this phase are missing in the relevant literature—quite in contrast to common bridge design practice, where specific requirements are routinely considered. Furthermore, no satisfying approaches for strategy evaluation accounting for the whole range of performance aspects can be identified in the relevant literature. On the contrary, it is found that the current investigations of maintenance strategies usually focus only on functional and economic aspects. In exceptional cases, where societal, cultural and environmental issues are explicitly considered during maintenance planning, these aspects are usually treated in monetary terms. This method, however, results in a significant loss of transparency and representativeness due to debatable pricing methods for nonmonetary aspects and trade-off effects. Nonmonetary transformation functions such as point rating systems, which are often used within sustainability analyses, show analogous shortcomings.

In summary, the following four issues turn out to be worth further investigation:

- provision of an empiric evidence for bridge specific requirements and maintenance
- development of practical tools for the requirements/objectives development phase
- development of an approach to bridge specific multi-objective maintenance planning
- experimental application of the approach for gaining insight into the influence both of the bridge specific context and of the manifold performance aspects on strategic maintenance planning decisions

## 6.3 Evidence for Bridge Specific Requirements and Maintenance Application

An indication for the existence of bridge specific requirements is found by the survey study of 24 bridges in Hamburg (Germany) and in Chennai (India) as described in chapter 3 of the present thesis. This study uses a questionnaire, which had been developed for this survey together with a data analysis method. The questionnaire is specifically tailored to identify bridge specific stakeholders' demands, owners' requirements and the bridge managers' maintenance objectives. It also identifies characteristic patterns of bridge specific maintenance strategies.

The survey indicates that the investigated bridges can be differentiated by their bridge specific profiles of the stakeholders' demands, of the owners' requirements, and of the maintenance objectives. It shows that bridges with similar surroundings, usage, and perception show similar patterns. By this, a first empiric evidence for the claimed hypothesis of bridge specific requirements can be provided.

The investigation shows that neither in Hamburg nor in Chennai structured approaches for maintenance strategy assessment and evaluation are applied. While in Chennai one uniform strategy seems to be applied to all bridges, in Hamburg specific maintenance strategies for different bridges can be made out. However, it appeares that the bridge specific maintenance application schemes in Hamburg result only from informal consideration of bridge specific peculiarities. From this it can be concluded that structured approaches for bridge specific maintenance strategy assessment and evaluation will constitute a stringent improvement to current bridge management practice in both cities.

In summary, the underlying hypothesis of bridge specific requirements is put on firmer ground. It becomes evident that the current practice of bridge management can draw benefit from the development of a new approach that allows for maintenance planning under consideration of bridge specific preferences.

## 6.4 New Tools for the Requirements/Objectives Development Phase

The above mentioned questionnaire is a suitable tool for gaining promising results within the requirements/objectives development phase. It identifies the bridge specific stakeholders' demands as well as the owners' requirements and the bridge managers' maintenance objectives.

Additional tools are two innovative performance indicators, which serve as a response to deficiencies in the common representation of societal, cultural and environmental issues. One of these indicators, *average availability* ($A_{av}$), represents the average traffic capacity during the service life of a bridge. It describes the average ratio of the actual traffic flow to the design traffic capacity. In doing so it captures the societal implications of the in-service behaviour of a bridge, with its primary function as a traffic link. As distinct from common approaches, this is done without the distortion by monetarisation of detour times and traffic accidents, which is commonly used to express traffic-related implications. The other newly developed indicator, *relative time of unsatisfactory appearance* ($RT_{ua}$), describes the fraction of the service lifetime, in which the condition of the concrete surface is below a specifically defined aesthetic threshold. This indicator provides a measure for the gradual changes of the aesthetic quality of a bridge. By this, it covers a field, which up to now has been widely neglected by the relevant literature.

## 6.5 Approach to Multi-Objective Maintenance Planning

The approach to multi-objective maintenance planning was subject of the most extensive sub-investigation within the present thesis. Chapters 4 and 5 portray the development of the SAMS-program for *simulation based assessment of maintenance strategies* and its example application on two case study bridges. In this thesis, the SAMS program is effectively combined with the method of *qualitative preference/performance-profile comparison*. Together with the described questionnaire, this set of instruments constitutes an innovative approach for multi-objective maintenance planning. This approach embraces:

- identification of the bridge specific performance requirements and objectives with the help of the questionnaire
- modelling of the bridge structure as well as of the deterioration processes, the maintenance activities, and their interactions
- assessment of alternative maintenance strategies (simulation-based)
- evaluation of the assessment results with reference to the preference profiles, which have been identified with the help of the questionnaire
- selection of the most appropriate maintenance strategy—optionally after iterative strategy improvement

The SAMS-program establishes the prerequisite for thorough multi-objective maintenance planning. It allows for the consideration of a wide range of performance indicators, which represent the whole spectrum of functional, economic, societal and cultural as well as environmental aspects. Even if the latter aspect is not considered in the example application in chapter 5, appropriate indicators, such as *$CO_2$-equivalen emissions*, are already at hand from the relevant literature and can be quite easily introduced in the SAMS-routine.

In addition, the method of *qualitative preference/performance-profile comparison*, which is introduced in chapter 2 of this thesis, allows for proper multi-objective evaluation of the performance results. With the help of the above described questionnaire, it overcomes the shortcomings of the widely spread single-unit transformation—including monetarisation—or simple dominance analysis. Hence, this method, with its unprecedented application in the field of bridge maintenance planning, both contributes to research and promises new impulses to practical maintenance planning.

Moreover, the SAMS-program allows for profound bridge specific maintenance planning. The detailed representation of the bridge structure, of the deterioration processes, of the maintenance activities, and of their manifold interactions enables the program user to consider peculiarities to a great extent. The structural representation, for example, takes into account the local variations of the internal forces and the geometric dimensions as well as for the variations of the mechanical properties. Likewise, deterioration mechanisms refer to specific mechanical and geometric properties. In addition, the representation of maintenance activities includes—besides maintenance costs—effects on the deterioration progress and the traffic flow. Finally, the possibility to adapt input data to a specific bridge structure by taking into account the findings from bridge assessment activities completes the bridge specific character of this planning instrument. By this, the SAMS-program allows for unprecedented precision: While Pommerening et al. (2008) apply a highly detailed consideration of the bridge condition by including details such as expansion joint and pavement condition and even account for location-dependent deterioration intensities, they leave out the structural safety. Likewise, Radojicic (2002) and Frangopol and Estes (1997), who treat structural safety and reliability in detail, do not consider other condition developments. Other approaches may consider structural safety, condition and even traffic effects—however, this is done on a generalised level which is not sufficient to account for bridge specific peculiarities.

## 6.6 Experimental Application of the Approach

The results from an experimental application of the approach on two case study show that each maintenance strategy has its individual strengths, but at the same time has characteristic shortcomings, which let it fall back behind other strategies. The approach is successful insofar as it identifies strategies, which promise to be superior in helping the bridge fulfil its specific requirements.

The approach captures influences, which moderate or even decisively control the outcomes of maintenance strategies—especially influences that are hidden by the complexity of the system and otherwise may stay unrecognised. The comparison of the results of the two case study bridges shows that the bridge specific context does influence the strategy performance results. Specifically, it reveals that for the Brücke Bahrenfelder Chaussee the *proactive repair* *strategy* and the *preventive-proactive strategy* lead to relatively weaker results with respect to service life and maintenance costs than in the case of the Köhlbrandbrücke. These inconsistent performance patterns in inter-bridge comparison arise from structural peculiarities of the two bridges, which enabled the strategies to play out one of their specific strengths only in the case of the Köhlbrandbrücke.

In conclusion, the newly developed approach allows for truly multi-objective and bridge specific maintenance planning. It surpasses common approaches for two reasons: On the one hand, it reveals particular strengths and weaknesses of maintenance strategies arising from the bridge specific context. On the other hand, it helps to identify the optimal strategies to fulfil the bridge specific requirements.

## 6.7 Practical Implications and Outlook for Future Research

From the results of the present investigation, the following practical implications can be deduced:

- Bridge maintenance planning regularly faces the problem of multi-objective decision making with regard to a broad range of performance aspects—functional, economical, societal, cultural, and environmental. The planning task consequently includes the recognition of the various requirements as well as the multi-objective assessment and evaluation of maintenance strategies.
- Bridge maintenance planning should always take into account the bridge specific context—both in terms of the particular requirements for the in-service behaviour of the bridge and in terms of the factors that eventually determine this behaviour. The latter can be found especially in form of the structural layout of a bridge, its exposition to damaging influences, and both the specific efficacy and costs of the respective maintenance activities.
- For mastering the planning task, the present thesis proposes a consistent set of innovative instruments:
  - a questionnaire, which can be used to identify bridge specific requirements
  - the SAMS-program for bridge specific and multi-objective strategy simulation
  - a practicable approach for multi-objective strategy performance evaluation
  - an equally practicable approach for heuristic maintenance strategy improvement, which can be used for tailoring strategies to the bridge specific requirements

The following issues constitute promising fields for further research:

- Up to now, the newly developed questionnaire is used exclusively within the pilot study of the present investigation. Because it is a promising tool for identifying bridge specific requirements and maintenance objectives, its further development will be of much practical gain. Especially the criteria for identifying the stakeholders' demands with respect to traffic availability, aesthetic appearance, and preservation should be validated by further field studies—including direct questioning of bridge users and otherwise affected people. Furthermore, the identification of the owner's requirements and maintenance objectives should be diversified from single item to multi item questions.
- A promising improvement of the SAMS-program will be the development of a module for structural analysis. By this even better bridge specific model representation can be achieved. In addition, the effects of deterioration and maintenance activities on the internal forces, which specifically depend on stiffness-reducing effects of local deterioration, can be taken into consideration. Such a program upgrade could, for example, be implemented with the help of Matlab-based finite-element-analysis, such as the CALFEM-toolbox (CALFEM 2014).
- The further development of the multi-objective performance evaluation from *qualitative* profile comparison to a *quantitative* procedure will be of much gain. It allows for entirely automated strategy evaluation. For this purpose a quantitative expression for the likeness of the profile shapes would need to be developed.
- Finally, practical maintenance planning will benefit from further development of the heuristic strategy improvement. A possible area of investigation could be the implementation of automated optimisation procedures, such as genetic algorithms. Here, the above described quantitative procedure for strategy evaluating can be used for defining the objective function for optimisation.

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## Symbols and Notation

### Latin upper case letters

| | |
|---|---|
| $A_{av}$ | Average availability for traffic [–] |
| $A_p$ | Area of prestressing steel [mm²] |
| $A_{st}$ | Area of longitudinal strengthening [mm²] |
| $C_0$, $C_{0,owner}$ | Net present value of the total costs [€], NPV of the total *owner* costs [€] |
| $C_{app}$ | Application costs of a maintenance activity [€] |
| $CI$, $CI_0$ | Condition index [–], *initial* (*0*) condition index [–] |
| $CI_A$, $CI_{A,res}$ | Condition of the asphalt pavement (*A*), resulting overall condition of the asphalt pavement (*res*) [–] |
| $CI_{av}$ | Average bridge condition index [–] |
| $CI_{Bear}$, $CI_{Bear,res}$ | Bearing condition, *resulting* overall condition of the bridge bearings (*res*) [–] |
| $CI_{bridge}$, $CI_{bridge,av}$ | Resulting bridge condition, *average* resulting condition (*av*) [–] |
| $CI_{CS}$, $CI_{CS,res}$ | Condition of the concrete surface, *resulting* overall condition of the concrete surface (*res*) [–] |
| $CI_{CS,btm}$, $CI_{CS,kerb,}$ $CI_{CS,web}$, $CI_{CS,cant}$ | Condition of the concrete surface: bottom slab (*btm*), kerbs / sidewalks (*kerb*), web (*web*) and cantilever (*cant*) [–] |
| $CI_{CS,ua}$ | Threshold level for unsatisfactory concrete surface appearance [–] |
| $CI_{Drain}$, $CI_{Drain,res}$ | Condition of the drainage system, resulting overall condition of the drainage system (*res*) [–] |
| $CI_{Exp}$, $CI_{Exp,res}$ | Expansion joint condition, resulting overall condition of the bridge expansion joints (*res*) [–] |
| $C_{maint}$ | Costs of a maintenance activity [€] |
| $F_{Pd}$ | Design value of the prestressing force [kN] |
| $H$ | High intensity |
| $L$ | Low intensity |
| $M$ | Medium intensity |
| $M_{Rd}$ | Design value of the resistant moment [kNm] |
| $M_{Sd}$ | Design value of the bending moment [kNm] |
| $N$ | Total number, sample size [–] |
| $N_{Sd}$ | Design value of the longitudinal force [kN] |
| $Q$ | Concentrated variable life load (axle load) [kN] |
| $R_d$ | Code/standard-based strength |

| | |
|---|---|
| $RT_{UA}$ | Relative time of unsatisfactory appearance [–] |
| SAMS | Program for simulation based assessment of maintenance strategies |
| $S_d$ | Code/standard-based load-effect (e.g. bending moment) |
| $T_{app}$ | Time of application [a] |
| $T_i$ | Time to deterioration initiation [a] |
| $T_{SL}$ | Service life [a] |
| $V_{Rd}$, $V_{Rd,s}$ , $V_{Rd,c}$ | Design value of the resistance to shear [kN], shear resistance of the reinforcement (*s*) and of the concrete (*c*) |
| $V_{Sd}$ | Design value of the shear force [kNm] |
| $V_{Sd,V+T,rep}$ | Design value of the resulting shear force (from shear and torsion) in the representative web of a multi-web girder [kN] |

**Latin lower case letters**

| | |
|---|---|
| $a$ | Traffic availability [–], annuity [€] |
| $a_{st}$ | Area of shear strengthening [mm²/m] |
| $a_{sw}$ | Area of mild steel shear reinforcement [mm²/m] |
| $app_{max}$ | Maximum number of repeated maintenance activity application [–] |
| $b_{deck}$, $b_{btm}$, $b_w$ | Width of the bridge deck (*deck*), of the bottom slab (*btm*), and the web (*w*) [m] |
| $c$ | Cover concrete |
| $d_p$ | Effective depth to the prestressing reinforcement [m] |
| $d_t$. $d_{t1}$. $d_{t2}$ | Duration of the temporal reduction of the deterioration progress [a] |
| $d_{traf}$ | Duration of traffic flow reduction |
| $f_{cd}$, $f_{yd}$, $f_{pyd}$ | Design values of the concrete strength (*c*) and of the yield stress of mild (*y*) and prestressing steel (*py*) [MPa] |
| $f_{ck}$, $f_{yk}$, $f_{pyk}$ | Characteristic values of the concrete strength (*c*) and of the yield stress of mild (*y*) and prestressing (*py*) steel [MPa] |
| $f_e$ | Effective compressive strength of concrete [MPa] |
| $g$ | Dead loads |
| $h$, $h_{deck}$, $h_{btm}$ | Height [m], height of the bridge deck (*deck*) and of the bottom slab (*btm*) |
| $i$ | Discount rate [%] |
| k | $10^3$ (e.g. in k€: 1000 €) |
| m | $10^6$ (e.g. in m€: 1,000,000 €); however, it is also used to express $10^{-3}$ in connection with technical or geometrical dimensions, such as 1 mm = 1/1000 m. |

| | |
|---|---|
| min, mod, max | Minimum, modal, and maximum value of a distribution |
| $n$ | Number |
| $q$ | Uniformly distributed life loads [kN/m²] |
| $q_{torsional}$ | Uniformly distributed life loads ($q$) arranged in such a way that maximum torsional effects occur [kN/m²] |
| $t$, $t_0$ | Time, starting time (0) [a] |
| $u(x)$ | Value function with respect to a quality $x$ [–] |
| $w$, $w_{Bear}$, $w_{Exp}$, $w_{Drain}$ | Weighting factor, weighting factors for bearings (*bear*), expansion joints (*Exp*), and the drainage system (*Drain*) [–] |
| $x_i$ | Indicator, quality |
| $z$ | Effective shear depth |

**Greek letters**

| | |
|---|---|
| $\alpha$ | Deterioration rate [$a^{-1}$] |
| $\beta$ | Reliability Index [–] |
| $\Delta\delta$ | Incremental condition improvement [–] |
| $\delta$, $\delta_k$, $\delta_n$ | Deterioration level, deterioration level that marks a change of the deterioration rate ($k$), and the $\delta$-value to which the condition is set after intervention ($n$) [–] |
| $\varepsilon$, $\varepsilon_x$, $\varepsilon_1$ | Strain, strain in mid-depth of a cross-section ($x$), principal strain |
| $\eta$, $\eta_M$, $\eta_{V+T}$ | Degree of fulfilment, degree of fulfilment with respect to bending ($M$) and with respect to shear and torsion ($V$+$T$) [–] |
| $\theta$ | Stress band inclination [°] |
| $\mu$ | Mean value |
| $\nu$, $\nu_1$, $\nu_2$ | Temporal reduction of the deterioration progress [–] |
| $\nu_{traf}$ | Traffic flow reduction by maintenance application [–] |
| $\sigma^2$ | Variance |
| $\sigma$, $\sigma_c$, $\sigma_{st}$ | Standard deviation, stress, stress in concrete ($c$) [MPa], stress in strengthening material ($st$) [MPa] |
| $\psi_P$ | Tendon inclination [°] |

**Indices**

| | |
|---|---|
| 0 | Initial |
| 1, 2, 3, … | Counter |
| *A* | Asphalt pavement |
| *Av* | Average |
| *App* | Application |

| | |
|---|---|
| *Bear* | Bearing |
| *Btm* | Bottom, bottom slab |
| *C* | Construction, Concrete |
| *Cant* | Bridge deck cantilever |
| *CS* | Concrete surface |
| *d* | Design |
| *Drain* | Drainage system |
| *Elem* | (SAMS-) Element |
| *e*, eff | Effective |
| *G* | Limit-state, dead load |
| *i* | Counter |
| *Kerb* | Kerbs and sidewalks |
| *k* | Characteristic |
| *M* | Bending |
| *owner* | (Bridge-) owner |
| *P* | Prestressing |
| *R* | Resistance, Strength |
| *res* | Resulting |
| *S* | Load-effect |
| *s* | (Mild-) steel |
| *St* | Strengthening material |
| *T* | Torsion |
| *V* | Shear force |
| *web* | Web |

**Abbreviations**

| | |
|---|---|
| AAR | Alkali-aggregate reaction |
| FRP | Fibre-reinforced polymer |
| NPV | Net present value |
| spl. | Special |
| w.r.t. | With respect to |

# Appendix

# Appendix I

**QUESTIONNAIRE**

| **Section 1: Basic Information** |
|---|

**Name of the bridge:**…………………………………......................………………

**Overall area of deck** (length of bridge × width of deck):………….........................

**Year of completion**:……………………………….……………………………........…

**Traffic route names:** traffic route on the bridge / traffic route crossed by the bridge:

…………………………..........…..............……/….....................................................……

**Traffic route on the bridge:**

- ☐ Road
  - ☐ within urban area: mainly local passenger traffic and usual commercial/industrial traffic (no motorway/expressway type of traffic route)
  - ☐ outside of urban area, (e.g. rural road or country road, no motorway/expressway type of traffic route)
  - ☐ motorway/expressway type of traffic route
  - ☐ other:……………………………….
- ☐ Rail
  - ☐ local passenger traffic
  - ☐ long-distance passenger traffic
  - ☐ rail cargo
  - ☐ other:……………………………….

**The bridge can best be described as (please tick only one option):**

- ☐ landmark / impressive bridge
- ☐ city bridge
- ☐ site of cultural or historic interest / heritage site
- ☐ remote main traffic link (e.g. motorway/expressway outside of urban areas)
- ☐ remote low traffic link (e.g. rural road or country road)
- ☐ other: ………………………………………….

**Section 2: General Importance of the Bridge**

This section refers to data regarding three important aspects of the bridge:

- **Traffic**: Relevance of the bridge for the local traffic or the effects of (partial) closures of the bridge (detour and congestion of alternative routes)
- **Integration into environment:** Here is of special interest how intensively and with what interest the structure is perceived by local residents and wider sections of the population
- **Cultural Importance:** Importance due to cultural, historic or aesthetic specialities

<u>Please answer the following questions without extensive preparatory work —a classification of the bridge by tendency is fully sufficient.</u>

**Please evaluate the following aspects regarding the traffic:**

| | very high | high | medium | low | negligi-ble |
|---|---|---|---|---|---|
| The **traffic volume on the bridge** is (…) | ☐ | ☐ | ☐ | ☐ | ☐ |
| The **traffic volume within the surrounding road/rail network** is (…) | ☐ | ☐ | ☐ | ☐ | ☐ |
| The **importance of the bridge for the surrounding road/rail network** is (…) | ☐ | ☐ | ☐ | ☐ | ☐ |
| In case of (partial) closure of the bridge the **additional stress on alternative routes** is (…) | ☐ | ☐ | ☐ | ☐ | ☐ |
| The **detour** in case of (partial) closure of the bridge is (…) | ☐ | ☐ | ☐ | ☐ | ☐ |

**Please evaluate the following aspects regarding the integration into environment:**

**The distance from which a usual observer looks at the bridge is (...)**

- ☐ very close (you can see the surface in every detail)
- ☐ close (< 50m)
- ☐ medium (50 – 200m)
- ☐ far (you usually see the structure as a whole)
- ☐ very far

**A usual observer looks at the bridge (…)**

- ☐ very long and intensely, e.g. touristic site
- ☐ long
- ☐ medium (people dwell or spend time in the vicinity of the bridge)
- ☐ short (e.g. people just pass by the bridge)
- ☐ very short/not at all (e.g. small bridge over motorway outside of urban area)

**The bridge is located within the following environment:**

- ☐ within residential area or shopping area or other heavily frequented area
- ☐ within urban, but weakly frequented area
- ☐ outside of urban area but with some contact with the population
- ☐ far off urban area with minor contact with the population
- ☐ usually non-accessible to the population (e.g.. industrial area)

**In terms of publicity and its importance to the population it can be described as (…)**

(Please only tick the ‘highest‘ box which applies)

| …a symbol or landmark | …of suprare-gional interest | …of regional interest (e.g. city) | …of local inter-est (local resi-dents) | …purely func-tional |
|---|---|---|---|---|
| ☐ | ☐ | ☐ | ☐ | ☐ |

high ⟷ low

**Please evaluate the cultural importance of the bridge:**

Please consider to the following criteria for the evaluation of the cultural importance:

**Technical masterpiece**
- The technical realisation constitutes a (then) crucial innovation.
- The structure had exceptional size / dimensions for its time.
- The structure was built by a prominent engineer.
- The structure clarifies a known construction principle exceptionally well.

**Symbol for the historic or social development**
- The structure is connected to the history of a region.
- The structure is connected to a local legend or tradition.
- The structure has influenced the way of living within a region.

**Artistic value,** e.g. the structure stands out due to
- harmonic form
- good proportioning
- applied building materials
- recognised beauty

**Extraordinary integration into the environment**
- indispensable element of a place or a landscape

**Extraordinary historic value or scarcity value**
- relic or symbol for the way of living within a region
- structure, building materials or appearance can hardly be found somewhere else
- listed historic building / heritage site

Now, please indicate the level of agreement with the following statements:

| | perfectly applies | → → → → → → → | | | not true |
|---|---|---|---|---|---|
| | 5 | 4 | 3 | 2 | 1 |
| The bridge is a **technical masterpiece**. | ☐ | ☐ | ☐ | ☐ | ☐ |
| The bridge is a **symbol for the historic or social development.** | ☐ | ☐ | ☐ | ☐ | ☐ |
| The bridge has an **artistic value** | ☐ | ☐ | ☐ | ☐ | ☐ |
| The bridge shows **extraordinary integration into the environment** | ☐ | ☐ | ☐ | ☐ | ☐ |
| The bridge has **extraordinary historic value or scarcity value** | ☐ | ☐ | ☐ | ☐ | ☐ |

**Section 3: Bridge Requirements**

**For this bridge it is (*extremely important /……/ unimportant*) that …**

| | absolute priority | very important | important | not so important | (rather) unimportant |
|---|---|---|---|---|---|
| … the **costs** for maintenance and repair are low. | ☐ | ☐ | ☐ | ☐ | ☐ |
| …the **traffic capacity** is continuously high. | ☐ | ☐ | ☐ | ☐ | ☐ |
| … it shows a **good appearance** with little visible deterioration. | ☐ | ☐ | ☐ | ☐ | ☐ |
| … it reaches a very long **service life**. | ☐ | ☐ | ☐ | ☐ | ☐ |
| … the **environmental impact** (e.g. $CO_2$ emission or waste) is low. | ☐ | ☐ | ☐ | ☐ | ☐ |
| (***other***, *please specify*): ……….......................................……… | ☐ | ☐ | ☐ | ☐ | ☐ |

**Section 4: Maintenance Objectives**

**The applied maintenance strategy for this bridge has the following objectives:**

| | absolute priority | very im-portant | im-portant | not so im-portant | (rather) unimportant |
|---|---|---|---|---|---|
| low **costs** for maintenance and repair | ☐ | ☐ | ☐ | ☐ | ☐ |
| continuously high **traffic capacity** | ☐ | ☐ | ☐ | ☐ | ☐ |
| good **appearance** with little visible deterioration | ☐ | ☐ | ☐ | ☐ | ☐ |
| very long **service life**. | ☐ | ☐ | ☐ | ☐ | ☐ |
| low **environmental impact** | ☐ | ☐ | ☐ | ☐ | ☐ |
| (***other***, *please specify*): ……………………………… | ☐ | ☐ | ☐ | ☐ | ☐ |

## Section 5: Maintenance Application

**Please comment on the following statements:**

| | usually / perfectly applies | often / mostly applies | some-times / partly applies | excep-tional case | never / does not apply |
|---|---|---|---|---|---|
| Deterioration has reached high levels when necessary maintenance measures are applied. | ☐ | ☐ | ☐ | ☐ | ☐ |
| Measures are selected in such a way that the future development of deterioration will be affected in a favourable way. | ☐ | ☐ | ☐ | ☐ | ☐ |
| Special preventive measures are applied which prevent or slow down future deterioration (e.g. silane) | ☐ | ☐ | ☐ | ☐ | ☐ |

**Application of maintenance measures**

Please use the following terminology for answering the next questions.

**TERMINOLOGY**

- **critical** condition (action is overdue)
- **serious** condition (action is necessary to fulfil requirements)
- **poor** condition (action leads to substantial improvement of condition)
- **fair** condition (action is foresightedly planned in a mid-/long term)
- **satisfactory / good** or even better condition (action is taken early and preventively, e.g. in order to slow down deterioration or in combination with other measures)

**Example**: Concrete repair at 'poor condition' of concrete surface and rehabilitation of expansion joints at 'serious condition' of the joints.

| | critical | serious | poor | fair | satisfactory / good |
|---|---|---|---|---|---|
| Concrete repair | ☐ | ☐ | ☒ | ☐ | ☐ |
| Expansion joints repair | ☐ | ☒ | ☐ | ☐ | ☐ |

⇒ Please answer the questions on the **following page** on the basis of the above terminology.

Now, please specify the condition of the respective element at the time of application of the following maintenance measures.

| | critical | serious | poor | fair | satisfactory / good |
|---|---|---|---|---|---|
| Concrete repair (superstructure) | ☐ | ☐ | ☐ | ☐ | ☐ |
| Concrete repair (substructure) | ☐ | ☐ | ☐ | ☐ | ☐ |
| Rehabilitation of tendons | ☐ | ☐ | ☐ | ☐ | ☐ |
| Rehabilitation of kerbs | ☐ | ☐ | ☐ | ☐ | ☐ |
| Rehabilitation of asphalt pavement / surfacing | ☐ | ☐ | ☐ | ☐ | ☐ |
| Rehabilitation of drainage inlets | ☐ | ☐ | ☐ | ☐ | ☐ |
| Rehabilitation of drainage pipes | ☐ | ☐ | ☐ | ☐ | ☐ |
| Rehabilitation of expansion joints | ☐ | ☐ | ☐ | ☐ | ☐ |
| Rehabilitation of bearings | ☐ | ☐ | ☐ | ☐ | ☐ |
| Rehabilitation of steel structure | ☐ | ☐ | ☐ | ☐ | ☐ |
| Rehabilitation of corrosion protection (paint) | ☐ | ☐ | ☐ | ☐ | ☐ |
| Replacement of bridge / essential parts | ☐ | ☐ | ☐ | ☐ | ☐ |
| *other (please specify)*...................... | ☐ | ☐ | ☐ | ☐ | ☐ |

**Please state *if* and with what *intensity* the following typical <u>preventive</u> measures have been or will be applied to the bridge:**

Please use the following terminology for answering the next questions.

**TERMINOLOGY**

**never**: never has been applied and is not considered to be applied in future

**infrequently**: has been applied or may be applied in future, but not as often as would be necessary for good preventive effects

**regular**: has been applied or may be applied in future in such a way and in such time-intervals for good preventive effects

**intensive**: has been applied or may be applied in future in a very intensive way and in close time-intervals for excellent preventive effects

| | never | infrequently | regular | intensive |
|---|---|---|---|---|
| Treatment of concrete surface (hydrophobic treatment, coating, etc.) | ☐ | ☐ | ☐ | ☐ |
| Corrosion protection of steel (renewal of paining) | ☐ | ☐ | ☐ | ☐ |
| *other (please specify)*………… ……. | ☐ | ☐ | ☐ | ☐ |

**Please state *if* and with what *intensity* 'routine minor maintenance' (e.g. routine cleaning of deck and drainage inlets, minor patching) is applied:**

| | never | infrequently | regular | intensive |
|---|---|---|---|---|
| Routine minor maintenance application | ☐ | ☐ | ☐ | ☐ |

## INTERVIEW GUIDE

**Topic 1: General Framework of Maintenance Planning**

**Please describe shortly and in general terms your maintenance strategy (or strategies):**

..............................................................................................................................

**Following this, we would like to ask you the following detailed questions on the decision making process:**

- Which **objectives** do you pursue by maintenance planning (bridge stock / individual bridges) ?
- **Ranking** of objectives?
- **Impact of budget on** maintenance **planning decisions?**

**Topic 2: Maintenance Planning and Scheduling Practice**

- **Selection criteria:** By which criteria do you choose maintenance measures (selecting certain measures from all possible measures), e.g. condition, maintenance costs, traffic effects, aesthetic effects.
- **Prioritisation criteria**: Which prioritisation criteria do you apply for scheduling maintenance activities for a bridge (priority of activities)? Which prioritisation criteria do you apply for maintaining certain bridges prior to other bridges (priority of bridges)?
- **Scheduling** of maintenance measures: By what criteria do you schedule the application of maintenance measures (e.g. effects on traffic, seasonal particularities, technical parameters)?

**Topic 3: Assessment and evaluation of maintenance strategies**

- **Retrospective assessment and evaluation** of implemented strategies
  - Do you **assess / evaluate** the implemented maintenance strategies?
  - **How do you assess / evaluate** the strategies? Do you use particular **indicators** such as ‘overall condition of the bridge’ or ‘net present value of maintenance costs’ or ‘time of traffic disruption’?
- **Structured approaches for maintenance planning**
  - Do you compare **strategy alternatives** during planning of future maintenance works—for example by conduction variation studies?
  - If so, on what basis do you **compare the alternatives**? Do you apply specific **indicators**, such as bridge condition or maintenance costs?
  - Do you try to **optimise** or do you try to reach satisfactory results?

# Appendix II

The following tables provide detailed guidance for interpreting bridge inspection results in terms of the ‘1–0’ δ-scale, which is used in the scope of the present thesis for expressing the time-dependent condition of the following items:

- concrete surface
- asphalt pavement
- bearings
- expansion joints
- drainage system

**Concrete Surface**

As it is described in sub-section 4.4.5, the deterioration of the concrete surface occurs in the form of defects, which are categorised into four groups: *cracking*, *breaking*, *leakage related deficiencies*, and *dirt accumulation*.

**Rating scale for the condition of the concrete surface**

| δ-Rating | Cracking | Breaking | Leakages | Dirt accumulation |
|---|---|---|---|---|
| **1.00** (*excellent*) Negligible minor defects, negligible adverse effect on appearance | No, or only minor and negligible defects | | | |
| **0.75** (*good*) Minor local defects, minor adverse effect on appearance | Mostly local visible cracking | Minor (small defects) | Isolated and small leakages | In increasing intensity: weathering effects, bird droppings, graffiti, swoot deposits |
| **0.50** (*fair*) Mostly locally concentrated defects, adverse effect on appearance | Still mostly local visible cracking | Mostly local (small defects) | Bigger local leakages | |
| **0.25** (*poor*) Distributed defects, surface appears run-down | Distributed visible cracking | Distributed and heavier | Distributed leakages | |
| **0.00** (*very poor*) Widespread defects, surface appears heavily run-down | Widespread visible cracking | Widespread, very heavy | Heavy leakages | |

## Asphalt Pavement

The deterioration of the asphalt surface occurs in the form of defects, which are categorised into four groups: *surface defects*, *surface deformations*, *cracking*, and *potholes*.

**Rating scale for the condition of the asphalt pavement**

| δ-Rating | Surface defects [a] | Surface deformation [b] | Potholes | Cracking |
|---|---|---|---|---|
| **1.00** (*excellent*) Negligible minor defects, negligible adverse effect on traffic flow | No or only minor and negligible defects | | | |
| **0.75** (*good*) Minor local defects, minor adverse effect on traffic flow | Beginning ravelling or polishing, moderately rough surface | Rutting ≤ 1cm heavings ≤ 2cm | Potholes ≤ 2cm | Superficial cracks |
| **0.50** (*fair*) Distributed defects, adverse effect on traffic flow | Worn down surface with considerable loss of aggregate and binder | Rutting 1–3cm heavings 2–5cm | Potholes 2–5cm | Cracks extending through the pavement layers |
| **0.25** (*poor*) Distributed defects, pavement is run-down, high effect on traffic | — | Rutting > 3cm heavings > 5cm | Potholes > 5cm | — |
| **0.00** (*very poor*) Widespread defects, heavily run-down, major effects on traffic | — | very heavy and widespread | > 5cm and widespread | — |

[a] Typical surface defects are ravelling or polishing.

[b] Typical surface deformations are rutting, frost heavings, and shoving.

**Bearings**

The deterioration of the bearings can be described with respect to the aspects *integrity defects* and *restrictions to movements and rotation*. The following table lists example defects, which are based on the *Schadenskatalog* of the German RI-EBW-PRÜF (BMVBS 2007, 2013).

**Rating scale for the condition of the bridge bearings**

| **δ-Rating** | **Integrity defects [a]** | **Restricted movement** |
|---|---|---|
| **1.00** (*excellent*)<br>Negligible minor defects | No or only minor and negligible defects | |
| **0.75** (*good*)<br>Minor local defects | *R*: minor corrosion, dirt / debris accumulation<br>*S*: beginning corrosion<br>*E*: — | — |
| **0.50** (*fair*)<br>Heavier defects | *R*: corrosion, heavier dirt / debris accumulation<br>*S*: dirt accumulation on sliding plate, dislocation of sliding plate, loss of grease<br>*E*: ageing, cracking, dislocation | Limited movement |
| **0.25** (*poor*)<br>Heavier defects with imminent danger to structural safety | *R*: heavy dirt / debris accumulation, breaking of guidance components<br>*S*: —<br>*E*: heavy signs of ageing, cracking, dislocation | Heavily restricted or danger of blocking |
| **0.00** (*very poor*)<br>Heavy defects with effects on structural safety | R: breaking of roller<br>S: —<br>E: — | Blocked |

[a] The following abbreviations are used: *R* for roller bearing, *S* for sliding bearing, *E* for elastomeric bearing

**Expansion Joints**

The deterioration of the expansion joints can be described with respect to the aspects *integrity defects*, *restrictions to movements*, and *reduced ride comfort and user safety*. The following table lists example defects, which are based on the *Schadenskatalog* of the German RI-EBW-PRÜF (BMVBS 2007, 2013).

**Rating scale for the condition of the expansion joints**

| **δ-Rating** | **Integrity** [a] | **Movement** | **Comfort, user safety** |
|---|---|---|---|
| **1.00** (*excellent*)<br>Negligible minor defects, negligible adverse effect on traffic flow | No or only minor and negligible defects | | |
| **0.75** (*good*)<br>Minor defects, minor adverse effect on traffic flow | All types: dirt / debris accumulation, peeling paint, beginning corrosion, loose anchorage<br>*M*: loose or damaged seal<br>*A*: ageing material, cracking, depression of joint | No restrictions | All types: loosening components |
| **0.50** (*fair*)<br>Medium defects, adverse effect on traffic flow | All types: leakage, heavy corrosion, loose anchorage<br>*M:* water bypassing lamellas, loose or damaged seal, dislocated sliding bearings, toppled lamellas<br>*A*: cracking, depression of joint<br>*R*: loose sliding plates (but still in place) | Limited movement | All types: loose components, noisy operation |
| **0.25** (*poor*)<br>Heavy defects, high effect on traffic | All types: heavier leakage, broken anchorage<br>*M*: water bypassing lamellas, dislocation/loss of sliding bearings, toppled lamellas<br>*R*: loose sliding plates | — | — |
| **0.00** (*very poor*)<br>Critical defects, heavily run-down, major effects on traffic | *M*: broken lamellas<br>*R*: missing screws/bolts, bouncing sliding plates | — | — |

[a] The following abbreviations are used: *M* for modular/lamella joint, *A* for asphalt joint, *R* for roller-shutter joint

**Drainage System**

The deterioration of the drainage system can be described with respect to the aspects *integrity defects* and *clogging*. The following table lists example defects, which are based on the *Schadenskatalog* of the German RI-EBW-PRÜF (BMVBS 2007, 2013).

**Rating scale for the condition of the drainage system**

| δ-Rating | Integrity defects | Clogging |
|---|---|---|
| **1.00** (*excellent*)<br>Negligible minor defects, negligible adverse effects on function and structure | No or only minor and negligible defects | |
| **0.75** (*good*)<br>Minor local defects, minor adverse effect on function and structure | Pipe: leaks close to structural components, minor corrosion<br>Hangers: —<br>Inlets: — | No clogging<br>Inlets: dirt accumulation (no clogging) |
| **0.50** (*fair*)<br>Distributed defects, adverse effect on function and structure | Pipe: leaks close to structural components, heavy corrosion<br>Hangers: corrosion, loosening<br>Inlets: broken grate | Clogging of inlets |
| **0.25** (*poor*)<br>Distributed defects, drainage system is run-down, clogging | Pipe: leaks close to structural components, very heavy corrosion | Clogging of pipe |
| **0.00** (*very poor*)<br>Widespread defects, heavily run-down, pipe rupture | Rupture of pipes | — |

# Appendix III

## <u>Structural Representation of the Köhlbrandbrücke</u>

### Dimensions of the critical cross sections (Köhlbrandbrücke)

| **Elem.** | $h$ [m] | $b_{deck}$ [m] | $h_{deck}$ [m] | $b_{btm}$ [m] | $h_{btm}$ [m] | $b_{w,crit}$ [m] | $b_{w,res}$ [m] | $A_p$ [cm²] | $z_p$ [m] | $\psi_p$ [°] | $d_{st}$ [m] | $a_{sw}$ [cm²/m] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 117.5 | 3.7 | 18.0 | 0.325 | 7.0 | 0.35 | 0.6 | 1.2 | 397.5 | 3.33 | 0 | 3.30 | 15.4 |
| 101.5 | 3.6 | 17.0 | 0.325 | 6.0 | 0.35 | 0.6 | 1.2 | 347.6 | 3.35 | 0 | 3.20 | 15.4 |
| 01.5 | 3.0 | 17.0 | 0.25 | 9.0 | 0.25 | 0.55 | 1.1 | 329.2 | 2.68 | 0 | 2.70 | 19.9 |
| 57.5 | 2.0 | 5.6 | 0.3 | 2.8 | 0.2 | 0.4 | 0.4 | 79.2 | 1.72 | 0 | 1.80 | 15.4 |
| 118 | 3.7 | 18.0 | 0.325 | 7.0 | 0.35 | 0.9 | 1.8 | 397.5 | 1.8 | 10 | 3.30 | 26.7 |
| 117 | 3.7 | 18.0 | 0.325 | 7.0 | 0.35 | 0.9 | 1.8 | 525.5 | 0.27 | 10 | 3.30 | 24.0 |
| 101 | 3.6 | 17.0 | 0.325 | 6.0 | 1.3 | 1.0 | 2.0 | 1033.9 | 0.4 | 10 | 3.20 | 75.0 |
| 00 | 3.0 | 17.0 | 0.25 | 9.0 | 0.35 | 0.7 | 2.4 | 818.4 | 0.25 | 8 | 2.80 | 23.9 |
| 01 | 3.0 | 17.0 | 0.25 | 9.0 | 0.35 | 0.7 | 2.4 | 898.7 | 0.25 | 10 | 2.80 | 41.3 |
| 57.9 | 2.0 | 5.6 | 0.3 | 2.8 | 0.2 | 0.4 | 0.4 | 79.2 | 1.72 | 10 | 1.70 | 7.9 |

*Note: The values are simplifying adaptions of the design dimensions*

### FRP Strengthening (Köhlbrandbrücke)

| **Elem.** | $A_{st}$ [cm²] | $a_{st}$ [cm²/m] |
|---|---|---|
| 117.5 | 61.46 | 5.00 |
| 101.5 | 72.46 | 5.00 |
| 01.5 | 54.61 | 5.00 |
| 57.5 | 9.73 | 5.00 |
| 118 | — | 5.00 |
| 117 | 68.88 | 5.00 |
| 101 | 58.80 | 10.00 |
| 00 | — | 5.00 |
| 01 | 87.36 | 5.00 |
| 57.9 | — | 2.50 |

### Material properties (Köhlbrandbrücke)

| $f_{pyk}$ | $E_p$ | $f_{yk}$ | $E_s$ | $f_{ck}$ | $\sigma_{st}$ |
|---|---|---|---|---|---|
| 1420 | 200 | 420 | 210 | 35 | 1500 |

*Note*: All data in [MPa]. The material properties are assumed to be constant along the bridge. Except for the ultimate stress in the FRP-strengthening, all values are taken from Sigrist and Krahwinkel (2008).

### Internal forces (Köhlbrandbrücke)

| **Element** | $M_{Sd}$ [MNm] | $F_p$ [MN] | $V_{Sd,V+T,rep}$ [MN] |
|---|---|---|---|
| 117.5 | 123.90 | 23.20 | — |
| 101.5 | 141.03 | 20.00 | — |
| 01.5 | 89.70 | 21.38 | — |
| 57.5 | 10.58 | 4.99 | — |
| 118 | — | 28.92 | 3.79 |
| 117 | -131.15 | 22.90 | -5.18 |
| 101 | -322.12 | 53.00 | -11.48 |
| 00 | — | 66.53 | 4.00 |
| 01 | -209.69 | 67.08 | -5.43 |
| 57.9 | — | 4.99 | 1.13 |

*Note*: The values for the internal forces are based on the data by Sigrist and Krahwinkel (2008), however $V_{Sd,V+T,rep}$ is calculated in the scope of the present investigation under simplifying assumptions (see sub-section 4.3.3).

**Bridge equipment and concrete surface (Köhlbrandbrücke)**

| | Allocation | | | Weighting factors $w_i$ | | | |
|---|---|---|---|---|---|---|---|
| | $l$ | $N_{Bear}$ | $N_{Exp}$ | $w_{Bear}$ | $w_{Exp}$ | $w_{A/CS}$ | $w_{Drain}$ |
| **Element** | [m] | [pcs.] | [pcs.] | [–] | [–] | [–] | [–] |
| 117.5 | 428 | — | — | — | — | 0.32 | 1.00 |
| 101.5 | 620 | — | — | — | — | — | — |
| 01.5 | 1505 | — | — | — | — | 0.68 | — |
| 57.5 | 665 | — | — | — | — | — | — |
| 118 | — | 1 | 1 | 0.02 | 0.125 | — | — |
| 117 | — | 10 | 1 | 0.14 | 0.125 | — | — |
| 101 | — | 8 | 1 | 0.11 | 0.125 | — | — |
| 00 | — | 2 | 1 | 0.03 | 0.125 | — | — |
| 01 | — | 29 | — | 0.41 | — | — | — |
| 57.9 | — | 20 | 4 | 0.29 | 0.50 | — | — |
| Σ | 3218 | 70 | 8 | 1.00 | 1.00 | 1.00 | 1.00 |

**In-situ concrete strength tests (Köhlbrandbrücke)**

| **Year** | **Description** | | **Test results** [a] | | | $f_{c,m}$ |
|---|---|---|---|---|---|---|
| [—] | Location | Test method | [MPa] | | | [MPa] |
| 1972 [b] | All locations | Cube | 49 | | | 49 |
| 1986 [c] | Deck | Drilled cores | 31 | 34.9 | | 33 |
| 1988 [d] | Deck | Rebound hammer | 30.2 | | | 30.2 |
| 1992 [e] | Bottom slab | Drilled cores | 47.6 | 49.2 | | 48.4 |
| 1994 [f] | Deck | Drilled cores | 50 | 53.3 | 55.8 | 53 |
| 1997 [g] | Bottom slab | Drilled cores | 32 | 46.7 | | 39.4 |
| | Web | Drilled cores | 36.9 | 48.4 | 61.5 | 48.9 |
| | Cross beam | Drilled core | 45.1 | | | 45.1 |
| 2005 [h] | Cross beam | Drilled core | 53.3 | | | 53.3 |

[a] Test results are transformed to equivalent strength values of a cylindrical specimen with d=150mm. [b] Strength test during construction, Schwab and Homann (1975)
[c] Strom- und Hafenbau (1986d) [d] Strom- und Hafenbau (1988a)
[e] Strom- und Hafenbau (1993) [f] Wayss & Freytag (1994) [g] WTM (1997) [h] TUHH (2005)

**Data Estimates for the Maintenance Activities (Köhlbrandbrücke)**

| **Maintenance activity** | | **Costs** [a] [$10^3$ €] | **Traffic effect** [b] ($\nu_{traf}$ [–]; $dt_{traf}$[a]) |
|---|---|---|---|
| Intensified routine maintenance | | 15.4 [c] (per year) | (0.002 ; 1) |
| Hydrophobic impregnation (girder) | - west | 170 [d] | (0.25 ; 4/12) |
| | - east | 291 [d] | (0.25 ; 8/12) |
| (kerbs/sidewalks) | - west | 32 [d] | (0.125 ; 2/12) |
| | - east | 67 [d] | (0.125 ; 4/12) |
| Coating concrete surface (girder) | - west | 271 [d*] | (0.25 ; 0.5) |
| | - east | 489 [d*] | (0.25 ; 1.0) |
| (kerbs/sidewalks) | - west | 58 [d*] | (0.125 ; 0.375) |
| | - east | 121 [d*] | (0.125 ; 0.375) |
| Concrete repair (exterior girder surf.) | - west | 1193 [e] | (0.25 ; 0.5) |
| | - east | 2096 [e] | (0.25 ; 1.0) |
| Rehabilitation of kerbs/sidewalks | - west | 916 [e] | (0.25 ; 0.5) |
| | - east | 1875 [e] | (0.25 ; 1.0) |
| Pavement resurfacing | - west | 71 [f] | (0.25 ; 6/52) |
| | - east | 143 [f] | (0.25 ; 3/12) |
| Pavement replacement | - west | 649 [g] | (0.25 ; 3/12) |
| | - east | 1297 [g] | (0.25 ; 4/12) |
| Bridge deck reconstruction | - west | 775 [h] | (0.25 ; 1) |
| | - east | 1550 [h] | (0.25 ; 1.25) |
| Drainage system overhaul | | 1065 [e] | — |
| Expansion joint replacement | - Axis 119 | 126 [e] | (0.5 ; 1/52) |
| | - Axis 118 | 252 [e] | (0.5 ; 1/52) |
| | - Axis 100 | 126 [e] | (0.5 ; 1/52) |
| | - Axis 0 | 126 [e] | (0.5 ; 1/52) |
| | - Axis 15/30/42/50 | 624 [e] | (0.5 ; 1/52) |
| Expansion joint repair | - Axis 119 | 39 [e] | (0.5 ; 1/52) |
| | - Axis 118 | 39 [e] | (0.5 ; 1/52) |
| | - Axis 100 | 39 [e] | (0.5 ; 1/52) |
| | - Axis 0 | 39 [e] | (0.5 ; 1/52) |
| | - Axis 15/30/42/50 | 154 [e] | (0.5 ; 1/52) |

| Maintenance activity | | Costs [a] [$10^3$ €] | Traffic effect [b] ($\nu_{traf}$ [–]; $dt_{traf}$[a]) |
|---|---|---|---|
| | - Axis 100–0 | 22 [e] | (0.5 ; 2/365) |
| | - Axis 1–30 | 326 [e] | (0.5 ; 2/52) |
| | - Axis 31–42 | 217 [e] | (0.5 ; 1/52) |
| Bearing repair | - Axis 119–101 | 9 [i] | (0.5 ; 4/365) |
| | - Axis 100–0 | 1 [i] | (0.5 ; 1/52) |
| | - Axis 1–30 | 14 [i] | (0.5 ; 2/52) |
| | - Axis 31–42 | 10 [i] | (0.5 ; 1/52) |
| Bending resistance improvement | - 117.5 | 430 [j] | (0.25 ; 10/52) |
| | - 101.5 | 387 [j] | (0.25 ; 10/52) |
| | - 01.5 | 1290 [j] | (0.25 ; 30/52) |
| | - 57.5 | 774 [j] | (0.25 ; 20/52) |
| | - 117 | 430 [j] | (0.10 ; 20/365) |
| | - 101 | 387 [j] | (0.10 ; 20/365) |
| | - 01 | 2107 [j] | (0.10 ; 60/365) |
| Shear resistance improvement | - 118 | 22 [k] | (0.25 ; 2/52) |
| | - 117 | 220 [k] | (0.25 ; 18/52) |
| | - 101 | 198 [k] | (0.25 ; 2/52) |
| | - 00 | 32 [l] | (0.25 ; 2/52) |
| | - 01 | 928 [l] | (0.25 ; 29/52) |
| | - 57.9 | 640 [l] | (0.25 ; 20/52) |
| Tendon conservation | - 117.5 | 40 [m] | (0.25 ; 20/365) |
| | - 101.5 | 36 [m] | (0.25 ; 20/365) |
| | - 01.5 | 120 [m] | (0.25 ; 60/365) |
| | - 57.5 | 72 [m] | (0.25 ; 40/365) |
| | - 117 | 40 [m] | (0.25 ; 50/365) |
| | - 101 | 36 [m] | (0.25 ; 50/365) |
| | - 01 | 196 [m] | (0.25 ; 150/365) |

| Maintenance activity | | Costs [a] [$10^3$ €] | Traffic effect [b] ($\nu_{traf}$ [–]; $dt_{traf}$[a]) |
|---|---|---|---|
| Shear reinforcement conservation | - 118 | 4 [n] | (0.25 ; 2/52) |
| | - 117 | 40 [n] | (0.25 ; 18/52) |
| | - 101 | 36 [n] | (0.25 ; 2/52) |
| | - 00 | 4 [n] | (0.25 ; 2/52) |
| | - 01 | 116 [n] | (0.25 ; 29/52) |
| | - 57.9 | 80 [n] | (0.25 ; 20/52) |

[a] 1974-prices. The minimum and maximum value of the distributions are is assumed to 80% and 120% of the modal values

[b] Free estimate

[c] Estimated: 0.28 €/m² (per deck-surface, 2002-prices)

[d] Estimated: 10 €/m² + 100k€costs for special platform / scaffolding (treated surface, 2008-prices)

[d*] Estimated: 18 €/m² + 100k€costs for special platform / scaffolding (treated surface, 2008-prices)

[e] Derived from data provided by the HPA (2006, 2010) under consideration of the respective price-basis

[f] Estimated: 11 €/m² (pavement surface, 2008-prices)

[g] Estimated: 60 €/m² (pavement surface, 2008-prices)

[h] Derived from data provided by the HPA (2006) under consideration of the respective price-basis; however, the price seems low when compared to the kerbs/sidewalk rehabilitation works, for which the prices are also provided by the HPA (2010). The difference may be explained by the fact that the reconstruction work prices rely on completed works, while the kerbs/sidewalk rehabilitation prices are in fact estimates by the HPA for future works.

[i] Estimated: 1100 €/bearing × 2 bearings per axis × number of axes (2008-prices)

[j] Estimated: 100 k€/span (2008-prices)

[k] Estimated: 22 k€/pier (2 webs, 2008-prices)

[l] Estimated: 32 k€/pier (3 webs, 2008-prices)

[m] Estimated: 10 k€/span (2008-prices)

[n] Estimated: 10 k€/pier (2008-prices)

The maintenance-effect intensities (H, M, L) of each maintenance activity are assigned to the structural components, such as to the tendons in the webs or to the asphalt pavement, based on personal judgement:

**Maintenance effect intensities**

| Maintenance activity | H | M | L |
|---|---|---|---|
| Intensified routine maintenance | Expansion joints, pavement | Concrete strength (deck), bearings, drainage system, concrete surface (kerbs) | Mild steel (web), tendon (high + lowpoints), concrete strength (web), concrete surface (cantilever+ web), strengthening |
| Hydrophobic impregnation | | | |
| - Box girder | Concrete surface (webs+bottom) | Concrete surface (kerbs+cantilever), mild steel (webs), concrete strength (webs+bottom) | Tendon (lowpoints) |
| - Kerbs/sidewalks | Concrete surface (kerbs) | — | — |
| Coating (concrete surface) | | | |
| - Box girder | Concrete surface (webs) | Concrete surface (bottom +cantilever), | Tendon (lowpoints), mild steel (webs), concrete strength (webs+bottom) |
| - Kerbs/sidewalks | Concrete surface (kerbs) | — | — |
| Concrete repair (exterior girder surface) | Concrete surface (cantilever, webs, bottom) | Mild steel (webs), | Tendon (lowpoint), Concrete strength (web+bottom) |
| Rehabilitation of kerbs/sidewalks | Concrete surface (kerbs) | Concrete strength (deck) | — |
| Pavement resurfacing | Pavement | Concrete strength (deck) | Tendon (highpoint) |
| Pavement replacement | Pavement | Concrete strength (deck) | Tendon (highpoint) |
| Bridge deck reconstruction | Pavement | Tendon (highpoint) | Concrete strength (deck) |
| Drainage system overhaul | Drainage system | Tendon (lowpoint) | Concrete strength (bottom,web), mild steel (web) |

| Maintenance activity | H | M | L |
|---|---|---|---|
| Expansion joint replacement and repair | Expansion joints | Concrete strength (web, bottom, deck at joints), mild steel (web at joints), concrete surface (webs, bottom, cantilever, at joints) | Tendon (web at joints) |
| Bearing replacement and repair | Bearings | — | — |
| Bending resistance improvement | FRP-strengthening | — | — |
| Shear resistance improvement | FRP-strengthening | Mild steel (web) | — |
| Tendon conservation | Tendons | — | — |
| Shear reinf. conservation | Mild steel (webs) | — | — |

The maintenance-effect parameter values are partly based on personal judgement and partly derived from the observed condition development (see sub-sections 5.5.2 and 5.5.3).

**Maintenance activities: ν, dt, δ value-ranges [upper bound, modal value, lower bound]**

| Maintenance activity | H | M | L |
|---|---|---|---|
| Intensified routine maintenance | | | |
| $\nu_1$ | [0.15, 0.10, 0.05] | [0.05, 0.025, 0.01] | [0.01, 0.005, 0] |
| $dt_1$ | [10, 10, 10] | [10, 10, 10] | [10, 10, 10] |
| $\nu_2$ | — | — | — |
| $dt_2$ | — | — | — |
| $d\delta$ | — | — | — |
| $\delta_n$ | — | — | — |
| Hydrophobic impregnation | | | |
| $\nu_1$ | [1, 0.9, 0.8] | [1, 0.9, 0.8] | [0.75, 0.625, 0.5] |
| $dt_1$ | [4, 2, 0.01] | [4, 2, 0.01] | [4, 2, 0.01] |
| $\nu_2$ | [0.5, 0.4, 0.3] | [0.3, 0.18, 0.06] | [0.75, 0.5, 0.25] |
| $dt_2$ | [6, 6, 6] | [6, 6, 6] | [6, 6, 6] |
| $d\delta$ | — | — | — |
| $\delta_n$ | — | — | — |

| Maintenance activity | H | M | L |
| --- | --- | --- | --- |
| Coating (concrete surface) | | | |
| $\nu_1$ | [1, 1, 1] | [1, 1, 1] | [1, 0.875, 0.75] |
| $dt_1$ | [2, 2, 2] | [2, 2, 2] | [2, 1, 0] |
| $\nu_2$ | [0.5, 0.4, 0.3] | [0.3, 0.18, 0.06] | [0.75, 0.5, 0.25] |
| $dt_2$ | [18, 16, 14] | [18, 16, 14] | [14, 12, 10 |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | [1, 1, 1] | — |
| Concrete repair (exterior girder surface) | | | |
| $\nu_1$ | — | [1, 0.95, 0.9] | [0.75, 0.625, 0.5] |
| $dt_1$ | — | [3, 2, 1] | [4, 3, 2] |
| $\nu_2$ | — | [0.5, 0.25, 0] | [0.5, 0.25, 0] |
| $dt_2$ | — | [7, 5, 3] | [7, 6, 5] |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |
| Rehabilitation of kerbs/sidewalks | | | |
| $\nu_1$ | [—] (west)<br>[0.28, 0.24, 0.2] (east) | [0.5, 0.4, 0.3] | [0.25, 0.125, 0] |
| $dt_1$ | [12, 10, 8] (west)<br>[20, 17, 14] (east) | [8, 7, 6] | [6, 4, 2] |
| $\nu_2$ | — | [0.3, 0.15, 0] | [0.1, 0.05, 0] |
| $dt_2$ | — | [6, 4, 2] | [2, 1, 0] |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |
| Pavement resurfacing | | | |
| $\nu_1$ | — | [0.25, 0.125, 0] | [0.125, 0.0625, 0] |
| $dt_1$ | — | [2, 1, 0] | [2, 1, 0] |
| $\nu_2$ | — | [0.125, 0.0625, 0] | [0.0625, 0.0313, 0] |
| $dt_2$ | — | [4, 3, 2] | [4, 3, 2] |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |

| Maintenance activity | H | M | L |
|---|---|---|---|
| Pavement replacement | | | |
| $\nu_1$ | [1, 1, 1] | [1, 0.95, 0.9] | [0.75, 0.5, 0.25] |
| $dt_1$ | [2, 1, 0] | [8.5, 7, 5.5] | [5, 2.5, 0] |
| $\nu_2$ | [0.45, 0.4, 0.35] (west)<br>[0.25, 0.2, 0.15] (east) | [0.6, 0.5, 0.4] | — |
| $dt_2$ | [20, 20, 20] | [6.5, 5.5, 4.5] | — |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |
| Bridge deck reconstruction | | | |
| $\nu_1$ | [1, 1, 1] | [1, 0.95, 0.9] | [1, 0.9, 0.8] |
| $dt_1$ | [2, 1, 0] | [100, 25, 14] | [9, 8, 7] |
| $\nu_2$ | [0.45, 0.4, 0.35] (west)<br>[0.25, 0.2, 0.15] (east) | — | — |
| $dt_2$ | [20, 20, 20] | — | — |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |
| Drainage system overhaul | | | |
| $\nu_1$ | — | [0.75, 0.5, 0.25] | [0.5, 0.375, 0.25] |
| $dt_1$ | — | [5, 3, 1] | [5, 3, 1] |
| $\nu_2$ | — | [0.25, 0.125, 0] | [0.25, 0.125, 0] |
| $dt_2$ | — | [7, 5, 3] | [7, 5, 3] |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |
| Expansion joint replacement | | | |
| $\nu_1$ | — | [0.6, 0.5, 0.4] | [0.4, 0.2, 0] |
| $dt_1$ | — | [12, 10, 8] | [12, 10, 8] |
| $\nu_2$ | — | — | — |
| $dt_2$ | — | — | — |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |

| Maintenance activity | H | M | L |
|---|---|---|---|
| Expansion joint repair | | | |
| $\nu_1$ | [-0.75, -0.75, -0.75] (119)<br>[-1.33, -1.33, -1.33] (118)<br>[-0.17, -0.17, -0.17] (100+0)<br>[-0.89, -0. 89, -0. 89] (15–42) | [0.6, 0.5, 0.4] | [0.4, 0.2, 0] |
| $dt_1$ | [4, 4, 4] | [6, 5, 4] | [6, 5, 4] |
| $\nu_2$ | — | — | — |
| $dt_2$ | — | — | — |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |
| Bearing replacement | | | |
| $\nu_1$ | [0.38, 0.33, 0.27] (31–42)<br>[— ] (other elements) | — | — |
| $dt_1$ | [20, 20, 20] (31–42)<br>[— ] (other elements) | — | — |
| $\nu_2$ | — | — | — |
| $dt_2$ | — | — | — |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |
| Bearing repair | | | |
| $\nu_1$ | [0.77, 0.77, 0.77] (100–0)<br>[— ] (other elements) | — | — |
| $dt_1$ | [35, 35, 35] (100–0)<br>[— ] (other elements) | — | — |
| $\nu_2$ | — | — | — |
| $dt_2$ | — | — | — |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |
| Bending resistance improvement | | | |
| $\nu_1$ | — | — | — |
| $dt_1$ | — | — | — |
| $\nu_2$ | — | — | — |
| $dt_2$ | — | — | — |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |

| **Maintenance activity** | **H** | **M** | **L** |
|---|---|---|---|
| Shear resistance improvement | | | |
| $\nu_1$ | — | [1, 1, 1] | — |
| $dt_1$ | — | [10, 8, 6] | — |
| $\nu_2$ | — | [1, 0.75, 0.5] | — |
| $dt_2$ | — | [15, 12, 9] | — |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |
| Tendon conservation | | | |
| $\nu_1$ | [1, 1, 1] | — | — |
| $dt_1$ | [10, 8, 6] | — | — |
| $\nu_2$ | [0.75, 0.5, 0.25] | — | — |
| $dt_2$ | [4, 4, 4] | — | — |
| $d\delta$ | — | — | — |
| $\delta_n$ | — | — | — |
| Shear reinforcement conservation | | | |
| $\nu_1$ | [1, 1, 1] | — | — |
| $dt_1$ | [4, 3, 2] | — | — |
| $\nu_2$ | [0.75, 0.5, 0.25] | — | — |
| $dt_2$ | [4, 3, 2] | — | — |
| $d\delta$ | — | — | — |
| $\delta_n$ | — | — | — |

**Structural Representation of the Brücke Bahrenfelder Chaussee**

### Dimensions of the critical cross sections (Bahrenfelder Chaussee)

| | $h$ | $b_{deck}$ | $h_{deck}$ | $b_{btm}$ | $h_{btm}$ | $b_{w,crit}$ | $b_{w,res}$ | $A_p$ | $z_p$ | $\psi_p$ | $d_{st}$ | $a_{sw}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Elem.** | [m] | [m] | [m] | [m] | [m] | [m] | [m] | [cm²] | [m] | [°] | [m] | [cm²/m] |
| 0.5 | 1.5 | 15.9 | 0.35 | 10 | 0.18 | 0.75 | 1.8 | 35.6 | 1.31 | 0 | 1.3 | 30.41 |
| 01M | 1.5 | 15.9 | 0.35 | 10 | 0.45 | 5.30 | 5.3 | 62.0 | 0.17 | 0 | 1.1 | 60.82 |
| 01V | 1.5 | 15.9 | 0.35 | 10 | 0.45 | 1.2 | 3.6 | 62.0 | 0.18 | 1 | 1.1 | 50.28 |
| 02 | 1.5 | 15.9 | 0.35 | 10 | 0.18 | 0.75 | 1.8 | 54.1 | 0.80 | 5 | 1.3 | 30.41 |

*Note: The values are simplifying adaptions of the design dimensions*

### FRP Strengthening (Brücke Bahrenfelder Chaussee)

| | $A_{st}$ | $a_{st}$ |
|---|---|---|
| **Elem.** | [cm²] | [cm²/m] |
| 0.5 | 117.4 | 5.00 |
| 01M | 33.54 | 5.00 |
| 01V | 41.18 | 5.00 |
| 02 | — | 5.00 |

The *material properties* are assumed to be constant along the bridge. Except for the ultimate stress of the FRP-strengthening, all values are taken from the documentation of the bridge Bahrenfelder Chaussee (LSBG 2010).

### Material properties (Brücke Bahrenfelder Chaussee)

| $f_{pyk}$ | $E_p$ | $f_{yk}$ | $E_s$ | $f_{ck}$ | $\sigma_{st}$ |
|---|---|---|---|---|---|
| 1420 | 200 | 420 | 210 | 30 | 1500 |

*Note*: All data in [MPa]. The material properties are assumed to be constant along the bridge. Except for the ultimate stress in the FRP-strengthening, all values are taken from the documentation of the bridge Bahrenfelder Chaussee (LSBG 2010)

The values for the *internal forces* have been calculated based on a simplifying beam model representation taking into account the haunched cross sections with their varying geometric properties with the help of the structural design software RStab. In this model, the prestressing effects are accounted for by means of deviation forces. The direct prestressing action—in the form of $M_{p,dir}=F_p \times z_p$ and $V_{p,dir}=F_p \times \sin\psi_p$ —is eliminated from the overall resulting internal forces before comparing these internal forces to the cross-sectional resistances.

**Internal forces (Brücke Bahrenfelder Chaussee)**

| **Element** | $M_{Sd}$ [MNm] | $F_p$ [MN] | $V_{Sd,V+T,rep}$ [MN] |
|---|---|---|---|
| 0.5 | 30.02 | 18.39 | 1.45 |
| 01M | -52.70 | 31.89 | 0.10 |
| 01V | -28.70 | 31.89 | 2.53 |
| 02 | 11.02 | 27.81 | 1.03 |

The bridge equipment and the visible concrete surface are accounted for by the following weighting factors

**Bridge equipment and concrete surface (Brücke Bahrenfelder Chaussee)**

| | **Allocation** | | | **Weighting factors $w_i$** | | | |
|---|---|---|---|---|---|---|---|
| | $l$ | $N_{Bear}$ | $N_{Exp}$ | $w_{Bear}$ | $w_{Exp}$ | $w_{A/Drain/CS}$ | $w_{cs}$ |
| **Element** | [m] | [pcs.] | [pcs.] | [–] | [–] | [–] | [–] |
| 0.5 | 60.69 | 3 | — | 1 | — | 1 | — |
| 01M | 0 | — | — | — | — | — | — |
| 01V | 0 | — | — | — | — | — | — |
| 02 | 0 | — | 1 | — | 1 | — | 1 |
| Σ | 60.69 | 3 | 1 | 1 | 1 | 1 | 1 |

## Maintenance Activities Brücke Bahrenfelder Chaussee

### Data Estimates for the Maintenance Activities (Bahrenfelder Chaussee)

| Maintenance activity | | Costs [a] | Traffic effect [b] |
|---|---|---|---|
| | | [$10^3$ €] | ($\nu_{traf}$ [–]; $dt_{traf}$[a]) |
| Intensified routine maintenance | | 0.22 [c] (per year) | (0.002 ; 1) |
| Hydrophobic impregnation | (girder+piers) | 17 [d] | (0.25 ; 6/12) |
| | (kerbs/sidewalks) | 0.7 [d] | (0.125 ; 2.5/12) |
| Concrete coating | (girder+piers) | 35.8 [e] | (0.25 ; 8/12) |
| | (kerbs/sidewalks) | 1.5 [e] | (0.125 ; 4/12) |
| Concrete repair (girder+piers) | | 354.8 [f] | (0.25 ; 9/12) |
| Rehabilitation of kerbs/sidewalks | | 22.5 [g] | (0.25 ; 6/12) |
| Pavement resurfacing | | 11.2 [h] | (0.25 ; 2.5/12) |
| Pavement replacement | | 31.1 [i] | (0.25 ; 6/12) |
| Bridge deck reconstruction | | 115.7 [j] | (0.50 ; 6/12) |
| Drainage system overhaul | | 51 [k] | — |
| Expansion joint replacement | | 31.2 [l] | (0.5 ; 1/52) |
| Expansion joint repair | | 7.2 [m] | (0.5 ; 1/52) |
| Bearing replacement | | 9.3 [n] | (0.125 ; 1/52) |
| Bearing repair | | 4.6 [o] | (0.125 ; 1/52) |
| Bending resistance improvement | - 00.5 | 51.8 [p] | (0.25 ; 2/365) |
| | - 01M | 51.8 [p] | (0.1 ; 2/365) |
| Shear resistance improvement | - 00.5 | 25.9 [q] | (0.25 ; 1/52) |
| | - 01V | 25.9 [q] | (0.25 ; 1/52) |
| | - 02 | 25.9 [q] | (0.25 ; 1/52) |
| Tendon conservation | - 00.5 | 17.3 [r] | (0.1 ; 2/365) |
| | - 01M | 17.3 [r] | (0.1 ; 2/365) |
| Shear reinforcement conservation | - 00.5 | 8.6 [s] | (0.25 ; 1/52) |
| | - 01V | 8.6 [s] | (0.25 ; 1/52) |
| | - 02 | 8.6 [s] | (0.25 ; 1/52) |

[a] 1970-prices. The minimum and maximum value of the distributions is assumed to 80% and 120% of the modal values.

[b] Free estimate (occasionally relatively long durations of the traffic effects compared to the Köhlbrandbrücke take into account the fact that the bridge is less important to traffic so that smaller maintenance teams and less strict time-schedules are expectably applied; also it must be considered that impregnation, coating, and concrete repair works also refer to the easier accessible substructure in contrast to the Köhlbrandbrücke).

[c] Estimated: 0.28 €/m² (per deck-surface, 2002-prices)

[d] Estimated: 10 €/m² + 25k€costs for special platform / scaffolding (treated surface, 2008-prices)

[e] Estimated: 18 €/m² + 25k€costs for special platform / scaffolding (treated surface, 2008-prices)

[f] Derived from data provided by the LSBG (2010) under consideration of the cost-estimates for sidewalk rehabilitation, concrete coating, pavement resurfacing and expansion joint replacement, which have been applied together with concrete repair works for overall costs of 1.769 mDM at 1988 prices

[g] Estimated to 570 €/m² (kerb surface, 2008-prices) on the basis of Pommerening et al. (2008: 36) with 100% surcharge do to the small area of the kerbs.

[h] Estimated: 11 €/m² (pavement surface, 2008-prices)

[i] Estimated: 60 €/m² (pavement surface, 2008-prices)

[j] Estimated: 120 €/m² (deck surface, 1989-prices)

[k] Estimated: 95 DM/m² (deck surface, 1985-prices)

[l] Estimated: 120 kDM (1988-prices), which is equal to 1887 €/m at 1988 prices

[m] Derived from data provided by the LSBG (2010)

[n] Estimated: 2250 €/ bearing (2008-prices)

[o] Estimated: 1100 €/bearing (2008-prices)

[p] Estimated: 100 k€/span (2008-prices)

[q] Estimated: 75 k€/pier (2008-prices)

[r] Estimated: 50 k€/span (2008-prices)

[s] Estimated: 25 k€/span (2008-prices)

**Maintenance Effect Intensities (Bahrenfelder Bridge)**

The maintenance-effect intensities (H, M, L) of each maintenance activity are assigned to the structural components, such as tendons in the webs or the asphalt pavement, based on personal judgement:

**Maintenance activities (effects on deterioration mechanisms)**

| Maintenance activity | H | M | L |
|---|---|---|---|
| Intensified routine maintenance | Expansion joints, pavement | Concrete strength (deck), bearings, drainage system, concrete surface (kerbs) | Mild steel (web), tendon (high + lowpoints), concrete strength (web), concrete surface (cantilever+ web), strengthening |
| Hydrophobic impregnation | | | |
| - box girder | Concrete surface (webs+bottom) | Concrete surface (kerbs+cantilever), mild steel (webs), concrete strength (webs+bottom) | Tendon (lowpoints) |
| - kerbs/sidewalks | Concrete surface (kerbs) | — | — |
| Coating (concrete surface) | | | |
| - box girder | Concrete surface (webs) | Concrete surface (bottom +cantilever), | Tendon (lowpoint), mild steel (webs), concrete strength (webs+bottom) |
| - kerbs/sidewalks | Concrete surface (kerbs) | — | — |
| Concrete repair (exterior girder surface) | Concrete surface (cantilever, webs, bottom) | Mild steel (webs), | Tendon (lowpoint), Concrete strength (web+bottom) |
| Rehabilitation of kerbs/sidewalks | Concrete surface (kerbs) | Concrete strength (deck) | — |
| Pavement resurfacing | Pavement | Concrete strength (deck) | Tendon (highpoint) |
| Pavement replacement | Pavement | Concrete strength (deck) | Tendon (highpoint) |
| Bridge deck reconstruction | Pavement | Tendon (highpoint) | Concrete strength (deck) |

| Maintenance activity | H | M | L |
|---|---|---|---|
| Drainage system overhaul | Drainage system | Tendon (lowpoint) | Concrete strength (bottom,web), mild steel (web) |
| Expansion joint replacement and repair | Expansion joints | Concrete strength (web, bottom, deck at joints), mild steel (web at joints), concrete surface (webs, bottom, cantilever – at joints) | Tendon (web at joints) |
| Bearing replacement and repair | Bearings | — | — |
| Bending resistance improvement | FRP-strengthening | — | — |
| Shear resistance improvement | FRP-strengthening | — | — |
| Tendon conservation | Tendons | — | — |
| Shear reinforcement conservation | Mild steel (webs) | — | — |

The maintenance-effect parameter values are partly based on personal judgement and partly derived from the observed condition development (see sub-sections 5.5.2 and 5.5.3).

**Maintenance activities: ν, dt, δ value-ranges [upper bound, modal value, lower bound]**

| **Maintenance activity** | **H** | **M** | **L** |
|---|---|---|---|
| Intensified routine maintenance | *Corresponds to the data for the Köhlbrandbrücke* | | |
| Hydrophobic impregnation | *Corresponds to the data for the Köhlbrandbrücke* | | |
| Coating (girders & substructure) | | | |
| $\nu_1$ | [1, 1, 1] | [1, 1, 1] | [1, 0.875, 0.75] |
| $dt_1$ | [3, 2, 1] | [3, 2, 1] | [4, 2, 0] |
| $\nu_2$ | [0.5, 0.4, 0.3] | [0.3, 0.18, 0.06] | [0.75, 0.5, 0.25] |
| $dt_2$ | [22, 20, 18] | [22, 20, 18] | [10, 8, 6] |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | [1, 1, 1] | — |
| Coating (sidewalks) | | | 05 |
| $\nu_1$ | [1, 1, 1] | [0.8, 0.7, 0.6] | [0.35, 0.2, 0.05] |
| $dt_1$ | [3, 2, 1] | [6, 4, 2] | [6, 4, 2] |
| $\nu_2$ | [0.5, 0.4, 0.3] | [0.5, 0.4, 0.3] | 0.35, 0.2, 0.05] |
| $dt_2$ | [0, 20, 20] | [12, 11, 10] | [17, 17, 17] |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | [1, 1, 1] | — |
| Concrete repair (exterior girder surface) | *Corresponds to the data for the Köhlbrandbrücke* | | |
| Rehabilitation of kerbs/sidewalks | | | |
| $\nu_1$ | — | [0.5, 0.4, 0.3] | [0.25, 0.125, 0] |
| $dt_1$ | — | [8, 7, 6] | [6, 4, 2] |
| $\nu_2$ | — | [0.3, 0.15, 0] | [0.1, 0.05, 0] |
| $dt_2$ | — | [6, 4, 2] | [2, 1, 0] |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |

| **Maintenance activity** | **H** | **M** | **L** |
|---|---|---|---|
| Pavement resurfacing | | | |
| $\nu_1$ | [0.06, 0.06, 0.06] | [0.25, 0.125, 0] | [0.125, 0.0625, 0] |
| $dt_1$ | [20, 18, 16] | [2, 1, 0] | [2, 1, 0] |
| $\nu_2$ | — | [0.125, 0.0625, 0] | [0.0625, 0.0313, 0] |
| $dt_2$ | — | [4, 3, 2] | [4, 3, 2] |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |
| Pavement replacement | | | |
| $\nu_1$ | [1, 1, 1] | [1, 0.95, 0.9] | [0.75, 0.5, 0.25] |
| $dt_1$ | [2, 1, 0] | [8.5, 7, 5.5] | [5, 2.5, 0] |
| $\nu_2$ | [0.25, 0.2, 0.15] | [0.6, 0.5, 0.4] | — |
| $dt_2$ | [20, 20, 20] | [6.5, 5.5, 4.5] | — |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |
| Bridge deck reconstruction | | | |
| $\nu_1$ | [1, 1, 1] | [1, 0.95, 0.9] | [1, 0.9, 0.8] |
| $dt_1$ | [2, 1, 0] | [100, 25, 14] | [9, 8, 7] |
| $\nu_2$ | [0.45, 0.4, 0.35] | — | — |
| $dt_2$ | [20, 20, 20] | — | — |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |
| Drainage system overhaul | *Corresponds to the data for the Köhlbrandbrücke* | | |
| Expansion joint replacement | | | |
| $\nu_1$ | [1, 1, 1] | [0.6, 0.5, 0.4] | [0.4, 0.2, 0] |
| $dt_1$ | [3, 2, 1] | [12, 10, 8] | [12, 10, 8] |
| $\nu_2$ | — | — | — |
| $dt_2$ | — | — | — |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |

| **Maintenance activity** | **H** | **M** | **L** |
|---|---|---|---|
| Expansion joint repair | | | |
| $\nu_1$ | [0.1, 0.07, 0.04] | [0.6, 0.5, 0.4] | [0.4, 0.2, 0] |
| $dt_1$ | [14, 14, 14] | [6, 5, 4] | [6, 5, 4] |
| $\nu_2$ | — | — | — |
| $dt_2$ | — | — | — |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |
| Bearing replacement | | | |
| $\nu_1$ | — | — | — |
| $dt_1$ | — | — | — |
| $\nu_2$ | — | — | — |
| $dt_2$ | — | — | — |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |
| Bearing repair | | | |
| $\nu_1$ | — | — | — |
| $dt_1$ | — | — | — |
| $\nu_2$ | — | — | — |
| $dt_2$ | — | — | — |
| $d\delta$ | — | — | — |
| $\delta_n$ | [1, 1, 1] | — | — |
| Bending resistance improvement | *Corresponds to the data for the Köhlbrandbrücke* | | |
| Shear resistance improvement | *Corresponds to the data for the Köhlbrandbrücke* | | |
| Tendon conservation | *Corresponds to the data for the Köhlbrandbrücke* | | |
| Shear reinforcement conservation | *Corresponds to the data for the Köhlbrandbrücke* | | |

# Curriculum Vitae

**Grischa-Christian Dette**

| | |
|---|---|
| Date of Birth | 26/03/1975 |
| Place of Birth | Berlin (West) |
| Nationality | German |
| Family Status | Married, two children |
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**Education and Employment**

| | |
|---|---|
| 1982–1986 | Grunewald Grundschule in Berlin |
| 1986–1995 | Evangelisches Gymnasium zum Grauen Kloster in Berlin<br>(01/93–06/93: St Bede's / Grey Coat Hospital School, England) |
| 1995–1996 | Military service (Fallschirmjägerbataillon 313, Varel i.O.) |
| 1996–2003 | Technische Universität Berlin<br>Bauingenieurwesen (civil engineering)<br>Degree: Diplom-Ingenieur |
| 2003–2006 | Porr Technobau Berlin<br>Site engineer |
| 2006–2011 | Technische Universität Hamburg-Harburg<br>Institut für Massivbau (Prof. Viktor Sigrist)<br>Research associate / PhD-student |
| 2010–2011 | Indian Institute of Technology Madras<br>Structural Engineering Division (Prof. B. Nageswara Rao)<br>DAAD-funded research exchange (10/2010–01/2011) |
| 2012– | Ed. Züblin<br>Technisches Büro Tiefbau<br>Project manager |