DOCTORAL THESIS

Development of an Optimised Condition Assessment Plan for Common Reinforced Concrete Bridges in Estonia

Sander Sein

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Declaration:
Hereby I declare that this doctoral thesis, my original investigation and achievement, submitted for the doctoral degree at Tallinn University of Technology has not been submitted for a doctoral or equivalent academic degree.

Sander Sein

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Optimeeritud seisukorra kontrolliplaani välja töötamine tüüpilistele Eesti raudbetoonsildadele

SANDER SEIN

KIRJASTUS

Contents

List of publications

The thesis is based on four publications which are referred to in the text as Paper I, Paper II, Paper III and Paper IV. The list of author’s publications based on which the thesis has been prepared is indexed by SCOPUS.

I **Sein, S.,** Matos, J. C., & Idnurm, J. (2017). *“Statistical analysis of reinforced concrete bridges in Estonia.”* The Baltic Journal of Road and Bridge Engineering, 12(4), 225–233.

II Kušar, M., Galvão, N., & **Sein, S.** (2019). *“Regular bridge inspection data improvement using non-destructive testing.”* Life-Cycle Analysis and Assessment in Civil Engineering: Towards an Integrated Vision – Proceedings of the 6th International Symposium on Life-Cycle Civil Engineering, IALCCE 2018, 2019, pp. 1793–1797.

III **Sein, S.,** Matos, J. C., & Idnurm, J. (2019). *“Uncertainty in condition prediction of bridges based on assessment method – A case study in Estonia.”* IABSE Symposium, Guimaraes 2019: Towards a Resilient Built Environment Risk and Asset Management – Report, 2019, pp. 1758–1765.

IV **Sein, S**., Matos, J. C., Idnurm, J., Kiisa, M. & Coelho, M. (2021). *“RC bridge management optimisation considering condition assessment uncertainties.”* Proceedings of the Estonian Academy of Sciences as a Paper, volume No. 70/2. 172–189.

Author’s contribution to the publications

Contributions to the Publications in this thesis are:

I Collection, preparation and analysis of historical data, development of the algorithm of principal component analysis, writing of the whole manuscript.

II Collection of input data for non-destructive testing methods, assessment of the criteria and different methods, oral presentation in the conference.

III Collection of background information, organization of benchmark testing, preparation and analysis of data, development of the degradation model and updating algorithm in MATLAB, writing of the whole manuscript, and oral presentation in the conference.

IV Collection of background information and input data, development of the overall framework and MATLAB algorithm of optimisation and writing the whole manuscript.

Introduction

Road infrastructure is one of the backbones of modern society that can provide economic growth and sustainable development of countries (Biondini & Frangopol, 2018). Therefore, the planning of road infrastructure-related works is essential, but those plans cannot be set in stone, because the long service life and high investment need requires a broad and long-term view of efficient solutions. People need infrastructure because it anticipates demographic and social change and thus the decisions need to be farsighted enough to capitalise on the benefits of future technological advances. Uncertainties around the future mean that investment plans must contain flexibility, but at the same time still retain enough clarity in their strategic aims (OECD/ITF, 2017). In addition to planning, maintaining such a network is also a challenging but needed task to ensure the sustainability and growth of the economy. Due to the limited budget, the management of infrastructure needs new assessment and decision-making strategies to maximise the benefits of the investments done in the past.

Asset management and decision-making strategies of networks are usually formulated as an optimisation problem with the life-cycle cost, performance (including condition), and network functionality being the most used objectives. For the optimal management of a network, an optimal decision should be made regarding the types of intervention within the investigated network under limited resources (Dong et al., 2014, Frangopol et al., 2017). It is also important to stress that an essential element of the decision-making process is the uncertainty as to whether the final decision will lead to the best outcome. This uncertainty comes from the fact that we cannot predict or model accurately the scenarios that will be derived from our decisions. Therefore, engineering is mostly about good enough decisions, grounded on dependable evidence and a scientifically justifiable derivation, and not concerned with correct decisions, since this concept is impossible to assess.

The assessment of existing infrastructure is becoming even more important as the network is increasing. The network should withstand future demands and know the performance of the infrastructure, a common understanding of assessment strategies is mandatory. The assessment process itself can be sophisticated and depending on the asset to be evaluated and the information to be obtained (Jensen et al., 2014). To reach a comparable inspection level and reliable condition assessment, integration procedures and degradation models should be also a part of a management system and meet the customer demand (Taffe, 2018). Significant advances have already been made in the performance modelling and decision analysis regarding the maintenance of deteriorating civil engineering systems. To predict the performance of structures, it is important to develop deterioration models, many models have been proposed based on Markov chains (Thompson et al., 2005, Kallen, 2007), linear or non-linear probability functions (L. C. Neves & Frangopol, 2005), neural networks (Steelman & Garcia, 2020), lifetime functions (Yang et al., 2006). Still, the results provided by the deterioration models are considered subjective as they are usually associated with a significant level of uncertainty. First, these models are based on inspection records, which, by themselves reveals a high level of uncertainty due to the subjectivity of the evaluation by the inspectors. However, it is also due to the natural variability of the deterioration process, which depends on a high set of factors, such as material quality, traffic levels, pollution levels, environmental conditions, structural typology, among others (Kallen & Van Noortwijk, 2006).

Previous works on the assessment of life-cycle, including maintenance planning, and optimal design of structural systems have been proposed (Frangopol, 2011a), Frangopol, 2019, Biondini & Frangopol, 2018, Biondini & Frangopol, 2015, Frangopol & Soliman, 2016) concentrate on the structural degradation mechanisms because it is essential to properly quantify the reliability-based performance of structural systems. The complexity of the target state phenomena involved (Taffe, 2018) and the little information available make it impossible to model the deterioration with high precision (Lounis & Madanat, 2012). In the context of reliability and life-cycle assessment, uncertainty analysis is used to better explain and support decision-making processes (Ditlevsen, 1982, Ditlevsen, 2003, ISO, 2006, Faber, 2005, Lloyd & Ries, 2007) and taking the uncertainty as a part of the performance indicator is essential for reliable condition assessment and rational intervention planning including maintenance, repair, or replacement of existing structures (Biondini & Frangopol, 2018).

Although the broad idea of using uncertainty as a part of performance assessment is the same in practice, there is a gap between the scientific approach and commonly used methods in bridge management. There are still authorities in Europe, including Estonia, where condition rating is used instead of quantification of structural degradation mechanisms for maintenance planning and asset management. In the bridge management of the Estonian Transport Administration, condition data from inspections is used as an input for economic decision-making or planning repair and strengthening measures (Minister of Economic Affairs and Infrastructure, 2018). Besides, the obtained data is partially used as input for load and resistance assessment. According to Phares et al. (2004), visual inspections are an unreliable method for the evaluation of infrastructure conservation states. Different inspectors under different conditions, evaluate significantly differently the conservation state of the infrastructure, introducing an additional source of uncertainty (Corotis et al., 2005). With the use of advanced methods in inspection and condition assessment, it should be possible to overcome the problem and previous works in the field of uncertainty have suggested that when the reduction of uncertainty is possible, it should also be integrated into life-cycle prediction with the results of inspection (Mori & Ellingwood, 1994, Kim & Frangopol, 2012, Soliman et al., 2013, Budelmann et al., 2012, Malerba, 2014, Papakonstantinou & Shinozuka, 2014, Omikrine Metalssi et al., 2015).

In addition to management under uncertainties, it is important to allocate limited resources efficiently to balance the cost and performance, for this purpose, multi-objective optimisation techniques have been proposed (Liu & Frangopol, 2006, Frangopol & Liu, 2007b, Frangopol & Bocchini, 2012, Kim et al., 2013, Dong et al., 2015, Biondini & Frangopol, 2015). These techniques quantify performance at the network level in a probabilistic way and integrate multi-criteria techniques for optimum maintenance and repair strategies to reduce the extent of damage of the network to the society, economy and environment. This formulation often leads to alternative maintenance solutions that represent an optimised trade-off among the conflicting objectives under consideration. The decision-makers are then able to select a compromise maintenance solution according to the preferred balance among these objectives.

Bortot et al. (2006) stated that to establish an optimal management strategy, bridge network performance should be maximised, and minimisation of probability of failure and life-cycle cost should be considered. As with most of the existing BMS software, optimisation is done in long-term view and the possible deterioration of bridge elements is considered as a part of condition rating, which is predicted using statistical models, where homogeneous Markov chain models are most frequent (Zambon et al., 2018). Unfortunately, the knowledge about the deterioration of specific models is not fully employed in these statistical models. Therefore, efforts have been performed to improve the accuracy of condition-based deterioration models to be implemented in BMS software. Alternative to Markov models, Ferreira (2018) proposed that a new classification system that incorporates more quantitative information from visual inspections where subjectivity can be reduced in the process of assessing the structural condition should be defined and although the classification system remained the same the subjectivity of collected information is quantified.

Scope and limitations

Bridge management and data-driven decision-making have been important topics in engineering for more than 20 years. In addition, the areas that have been investigated in the field have changed within the years and although some of the main findings have been introduced in the first chapter the scope of this work has been narrowed down with currently used condition assessment and non-destructive testing practices of reinforced concrete bridges. The input data is directly taken from the Estonian national road data bank, regular bridge inspection database, experts who have carried out non-destructive testing and collected during the investigation. The optimisation in the subject matter has been defined through the uncertainty of a measurand with the best-estimated value. The optimisation of condition control is to propose practically useable methods to make the condition assessment more accurate and time between regular inspections optimal.

The limitations of this work are related to condition and performance assessment, which are widely used terms in bridge management and in many approaches these two are connected to structural performance, which goes out of the scope of this research. Structural performance relates to reliability, safety, serviceability and other variables that are linked with risks or service life and commonly used in the bridge design phase. These variables are commonly used in the load-bearing capacity evaluation of existing structures through investigation of design documentation, load testing, monitoring, locating existing details, assessing damages and material properties (Rücker et al., 2006). Although the same parameters are also investigated in this work the main focus is on making the currently used condition assessment more optimal. Condition assessment is widely used by practitioners because it is relatively quick and easy to conduct. The downside of the condition assessment is the subjectivity that comes with visual assessment and this affects the decision-making related to intervention activities. Based on the common practice in Estonia, the current work is limited to the condition assessment related to intervention decisions and is not dealing with the structural performance of a bridge. The limitation is needed to draw more attention to the events that take place before and after the design or re-design of a structure and affect the overall condition control practice.

Novelties

The main novelties of the work are related to the different approaches to the problem of performance assessment, the addition of uncertainty as a quantitative performance indicator to qualitative assessment and the use of a simple optimisation model based on condition assessment instead of alternative and more complex methods to make it more feasible to practitioners. Most of the previous development work in the context of bridge management is concentrated on optimising the maintenance and repair strategies, taking the condition assessment as a part of the overall system, but since the input itself has a high level of uncertainty, then these approaches work only with accurate information. In the current approach, uncertainty is also used to ensure that the collected data and final decision complies with the current management system, but the performance assessment framework is integrated with the owner's needs using statistical data evaluation according to Guide to the Expression of Uncertainty in Measurement (Hässelbarth et al., 2006). The use of a simple optimisation approach considering overall performance assessment including degradation modelling and model updating should help to reduce the knowledge gap between how data is collected and decisions are made in scientific approaches and practice. The optimisation is done using only time and level of uncertainty and like in the optimisation of maintenance strategies, the main difficulty is to define optimal conditions, like what is the maximum permissible performance, what should be the maximum condition of a structure presenting this level of deterioration and what maintenance actions to consider (M. L. Neves et al., 2011).

In the local context, the investigated topics should increase the knowledge of uncertainties in performance assessment, improve the condition assessment and give a different angle to bridge management in Estonia. Main novelties:

- Statistical analysis of structural and non-structural elements.
- Calculation of more universal Markov chain based deterioration models and verification of the results.
- Development of deterioration model updating format and conversion matrix of non-destructive test results.
- A broader investigation of the uncertainties related to condition assessment.
- Evaluation of the current condition control system and proposal of more optimal scheduling.

Aim and objectives

The main aim of the research work is to introduce the uncertainty in performance assessment and propose a framework to optimise inspection scheduling for regular condition assessment of bridges. The goal of the optimisation is to keep the level of uncertainty under the desired level and maximise the time between assessments. Since visual inspections are considered subjective, then to improve the accuracy of performance assessment, selection of suitable non-destructive testing methods is investigated and the conversion matrix is proposed to simplify the implementation of additional assessment methods. To present the applicability of the overall research work, the most common reinforced concrete national road bridges in Estonia are statistically studied and additional information about visual and non-destructive assessment method is collected. Since the condition data have never been modelled with probabilistic models, then verification of degradation models is also one important objective. Other notable objectives of the thesis are:

- Description of current bridge management regulation of Estonia.
- Investigation of Principal Component Analysis and the importance of different bridge element groups.
- A suggestion of criteria for suitable non-destructive test method selection.
- Investigation of uncertainties in different assessment methods, including visual inspection and basic non-destructive testing methods.
- Investigation and use of stochastic Markov chain models and Bayesian inference procedures with uncertainties.
- Development of optimisation algorithm in MATLAB® to find optimal time interval between assessments keeping the uncertainty under the desired threshold.

Research methods

The preparation of the dissertation was based on books, reports, scientific publications, statistical information and legislative documents like standards or national regulations. Also, a great amount of information was obtained through discussions with experts working in the public sector or research field and construction-related information was collected during inspections. The research consists of two different areas including more practical performance assessment and theoretical decision-making and the optimisation was done in a stepwise manner – starting from the collection of historical information ending with presenting the applicability of the proposed framework (Figure 1).

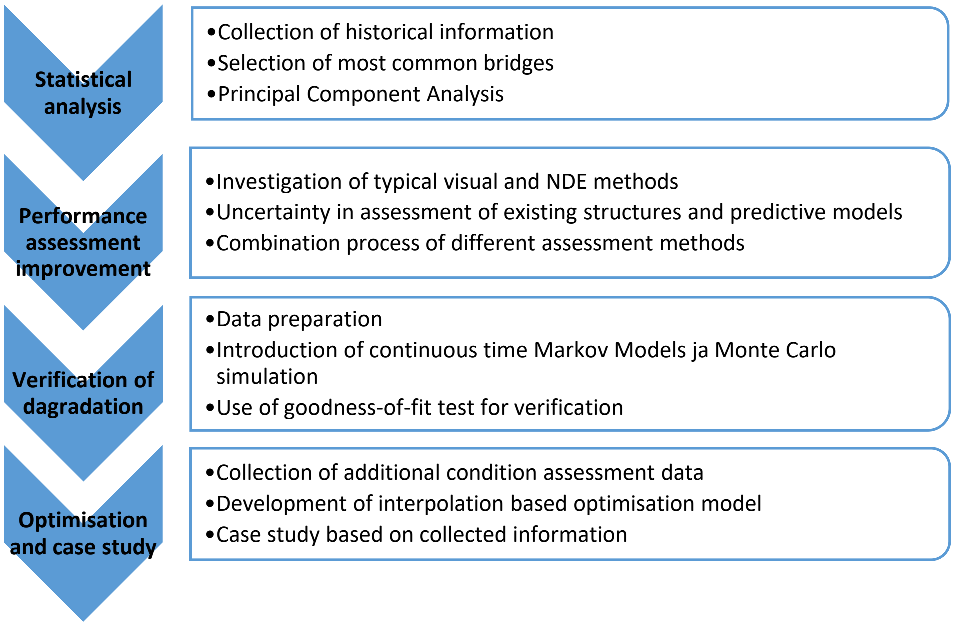

Figure 1. The stepwise procedure of optimisation.

In Paper I the collection of historical information and selection of most common bridges was done manually without any typical approach, but the statistical analysis was carried out using a multivariate method, called Principal Component Analysis (PCA). In addition to the analysis of principal components, the results of two different algorithms, Alternating Least Square (Ilin & Raiko, 2008) and Singular Value Decomposition (Chambers, 1977), were compared.

In Paper II the selection of non-destructive test method criteria and suitable methods, the Analytical Hierarchy Process technique (Saaty, 1990) and utility function was used to decompose a ranked list of variables.

In Paper III the obtained benchmarking results were compared using continuous-time Markov models (Kallen & Van Noortwijk, 2006) combined with the Monte Carlo method (Denysiuk et al., 2017) and Bayesian updating (L. C. Neves & Frangopol, 2008).

The overall concept of uncertainty related inspection scheduling was based on Taffe (2018) proposed approach and a case study was carried out using policies of the Estonian Transport Agency. The visual overview of the work is presented in Figure 2, where the relation between different topics, Publications and chapters can be seen.

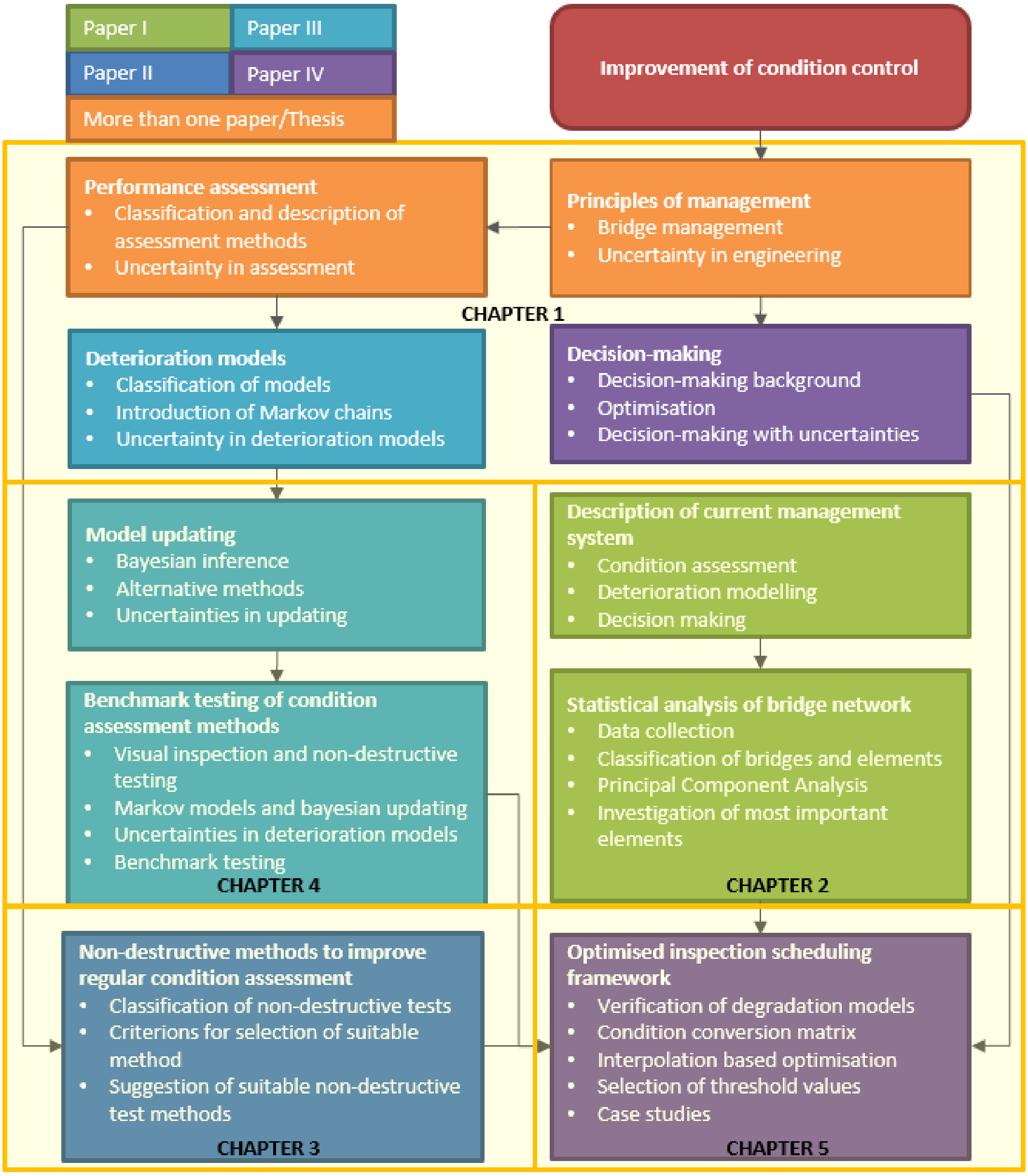

Figure 2. Research and publications overview.

The main results are compiled in Paper IV, where the degradation models were calculated and updated with the same methods as in Paper III, with additional verification based on the goodness-of-fit test under the assumption that the goodness-of-fit follows a χ_n^2 distribution (C. Ferreira et al., 2014).

Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
BMS	Bridge Management System
CS	Condition States of elements
CI	Condition Index of a bridge
CoV	Coefficient of Variation
FHWA	Federal Highway Administration
GUM	Guide to the expression of uncertainty in measurement
NBIP	National Bridge Inspection Program
NCHRP	National Cooperative Highway Research Project
NDT	Non-destructive testing
MR&R	Maintenance, Repair and Rehabilitation
MCDM	Multi-criteria decision-making
PCA	Principal Component Analysis

1 Bridge assessment and management principles

The dissertation concentrates on condition assessment, which is a part of a bigger systematic framework, widely known as bridge management system (BMS) and for a better understanding of the importance of uncertainty in this framework, main principles of BMS and management related uncertainties are introduced in a State-of-art literature review to help the reader to understand the current and broader situation of research done in the field of performance assessment.

1.1 Bridge management systems

A BMS is like any other infrastructure management system, that can be defined as a framework that coordinates its functions in an integrated, data-centred way to manage the physical system through its life-cycle, maintaining the elements at an adequate performance level (Grigg, 2012). It should be a systematic and rational approach including all activities related to managing a network of bridges, like optimisation of maintenance planning to maximise performance while minimising costs (Klanker et al., 2017).

BMS's have been developed since the late 1960s, when a series of bridge failures occurred and "motivated" the governments in the United States to mandate standard bridge inspection procedures (Thompson et al., 1998). Federal Highway Administration (FHWA) created the National Bridge Inspection Program (NBIP), to catalogue, record, and track in a database the state of all bridges located in the main road of the country. In the beginning, the role of the NBIP was only to inform the authorities about the state of the bridges and the necessities that they demanded in terms of maintenance actions in order not to reach a critical condition state. The interest in the development of a BMS only began to be visible from the 1980s, when the National Cooperative Highway Research Project (NCHRP) began a program to develop a model for an efficient BMS (Elbehairy, 2007). Based on the guidelines from the American Association of State Highway and Transportation Officials (AASHTO) published in 1993 any modern BMS should include the database, deterioration models, updating functions and optimisation model (Figure 1).

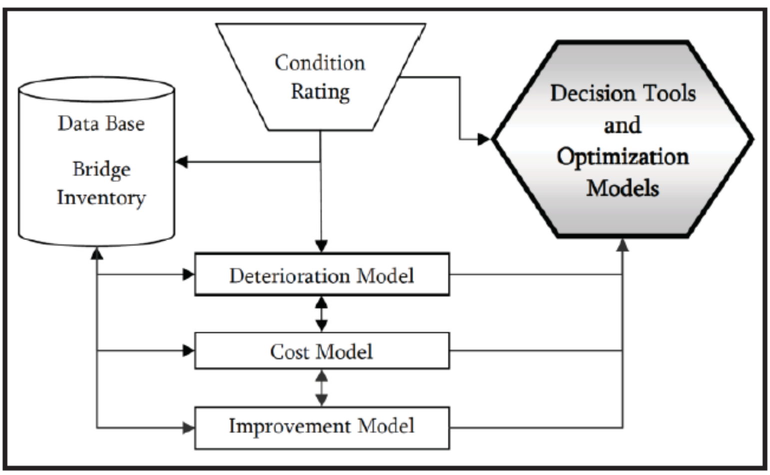

Figure 3. Basic components of classical BMS (AASHTO, 1993).

Basic components described in a more detailed form:

- **Data collection** – The database can be considered the heart of any management system. In addition to having the function of storing information such as name, location, age, among other characteristics of all infrastructures (inventory records), it should also store data on all inspections and maintenance actions performed in each infrastructure (inspection records). The database is not a static element, it must be regularly updated throughout the life-cycle of the infrastructures to keep update and to constitute a good historical record of the infrastructures (Elbehairy, 2007).
- **Deterioration model** – The deterioration models can be divided into three different groups: condition-based, damage process-based and reliability-based models. The first two have the main function of simulating the real degradation process of the infrastructures and the third is for probabilistic modelling of limit state. The main purpose is to assist the manager in making decisions about the actions to be performed on the structure. In general, the condition-based deterioration models can be based on inspection results, estimates obtained through expert opinion or by combining these two methodologies (Kallen, 2007). Damage process-based models are mostly analytical, based on measurements and used to model material properties. Reliability-based models are probabilistic and used for assessing the margin of safety or limit state function, the input can be design information or measurements (Ghosn et al., 2016).
- **Inference or update functions** – Management systems are related to continuous data flow with updating of the maintenance strategies and new information obtained from inspections. These should be updated frequently for two reasons. First, due to the uncertainty of forecasting models, whenever inspections are performed, an adjustment in performance forecasts occurs. Second, because often the optimal maintenance strategy is not followed, requiring the changes to correct these failures (M. L. Neves et al., 2011).
- **Optimisation or decision-making model** – The optimisation model has the task of defining the optimal Maintenance, Repair and Rehabilitation (MR&R) activities to be performed on the infrastructures, considering the constraints imposed in the management system and the results obtained from the deterioration model. In current models, these constraints are based on minimising maintenance costs with the structure, maximising structure performance throughout its life-cycle, and on some models, minimising the impact of maintenance actions on users (C. A. R. Ferreira, 2018). Also minimising the level of uncertainty of condition is related to optimal MR&R.

1.2 Uncertainty in bridge management and assessment

"*As far as the laws of mathematics refer to reality, they are not certain; and as far as they are certain, they do not refer to reality*." – Einstein, 1921.

Guide to the expression of uncertainty (JCGM, 2008) in measurement (GUM) defines, that uncertainty is a non-negative parameter that describes the dispersion of observations within the proximity of the best estimate. It also means doubt about the validity of the result of quantitative measurement (JCGM, 2008). For example, when a bridge condition is reported, also the best estimate of its value and the best evaluation of the uncertainty of that estimate should be given because it is not normally possible to decide in which direction the realistic condition of the bridge is and if the structure performs as intended.

Without the uncertainties, too much trust might be placed in the values reported and it may have undesired consequences, but on the other side, the overstatement of uncertainties could also have undesirable repercussions. For example, it could cause costly unnecessary interventions or users of measuring equipment to purchase instruments that are more expensive than they need.

From the management point of view, it is important to have a clear overview of the uncertainties and limited with a specified level of confidence and that satisfies one's needs. If the uncertainties are known, then in certain circumstances that could lead to a situation where it is necessary to exchange the data collection method with the one that provides an uncertainty that meets the needs. Based on GUM (2008), there are three distinct advantages to adopting an interpretation of the probability-based level of confidence, the standard deviation, and the law of propagation of uncertainty as to the basis for evaluating and expressing uncertainty in the assessment:

- The law of propagation of uncertainty allows the combined standard uncertainty of one assessment result to be used in the evaluation of the combined standard uncertainty of another assessment result or decision-making process.
- The combined standard uncertainty can serve as the basis for calculating intervals that correspond in a realistic way to their required levels of confidence.
- The frequent source of confusion of component classification, like "random" or "systematic", is unnecessary when evaluating uncertainty because all components of uncertainty are treated in the same mathematical way.

Based on the advantages the assessment result is always evaluated as a standard deviation of repeated observations (Type A) or available knowledge (Type B). Probability distribution based on available knowledge is used when the input quantity cannot be evaluated by an analysis of the results of an adequate number of repeated observations. Although there is common knowledge that Type A evaluations are more reliable than Type B evaluations, in many practical measurement situations where the number of observations is limited, the components obtained from Type B evaluations may be better known than the components obtained from Type A evaluations.

In bridge management, where decision-making normally leads to a specific outcome but when the actual result is unknown, then based on Yoe (2011) and Luce and Raiffa (1989), the probabilistic approach should be used and based on Holton (2004), the uncertainty in probabilistic is "a state of not knowing whether a proposition is true or false". From the management point of view, uncertainty is just the lack of exact knowledge, regardless of what is the cause of this deficiency (Refsgaard et al., 2007), but it may also result from a lack of a pattern in the system behaviour (*randomness*) (Blockley, 2012).

To utilise relevant information to provide the decision-maker with a realistic picture of the current knowledge of system behaviour over a given time, models that consider the uncertainty in the system performance and take the external conditions also into account should be used. Although there is no common knowledge on how to estimate uncertainties in any cases more complex than a well-known random process such as a coin toss it is still clear that relevant predictions require the appropriate understanding and management of uncertainty. For example, the description of the system performance plays a major role in most of the resistance analysis of engineering systems (Sánchez-Silva and Klutke, 2016) and to consider the uncertainties, safety factors are used. In bridge management, there are no commonly known safety factors and the visual inspection limits the overall efficiency to revealing the structural anomalies, which can lead to costly and inadequate maintenance actions to cover the uncertainty resulting from visual inspections.

Taffe (2018) listed different methods for condition assessment and proposed a procedure of how data would meet customer demand. Although the condition assessment is introduced as a method that should reveal the information of the inner structure, the issues regarding the definition of the measurand, identification of the method, location and timing of condition assessment are relevant components of uncertainty. The proposed procedure targets accuracy of the results to guarantee their reliability, which means that precision should be ensured with an uncertainty of the measurement, which should be statistically evaluated using GUM (JCGM, 2008) and the trueness has to be provided by well-trained personnel (Taffe, 2018). The main idea behind the quantification of knowledge is to identify the quantities influencing the results and allow one to draw reliable conclusions (Taffe and Gehlen, 2009). The conclusions should meet the minimum requirements of the client, not absolute minimal (Taffe, 2018).

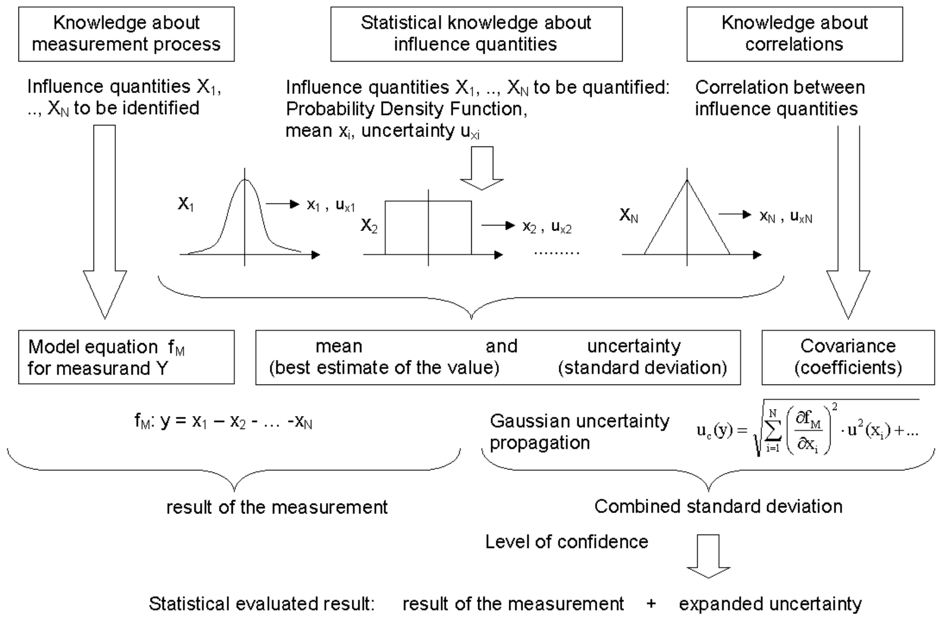

Figure 4. Flowchart of the measurement process and outcomes influencing the outcome. Adapted from Taffe (2018) and from Paper IV.

The result of the measurement process is decision-making, where information about the uncertainties related to each of the decision options estimates should be considered, as in most cases the certainty of the desired outcome of a decision is a central criterion (average value) on the selection of the management policy. On the other hand, any other decision may also include an increased probability of an extremely undesired outcome (such as the collapse of a bridge). If this information would be available, then the decision-maker may prefer to choose another decision option that reduces this risk even if the expected benefits would decrease as well (Uusitalo et al., 2015).

Uncertainty may come from various sources and can explicitly be distinguished by nature as aleatory or epistemic. Aleatory uncertainty comes from randomness or natural variability and epistemic uncertainty comes from incomplete knowledge and information (Kiureghian & Ditlevsen, 2009). In more detail, the uncertainty can be divided into 6 categories, which is based on the classification from ecology and conservation biology (Regan et al., 2002) and described by Uusitalo et al. (2015).

- **Inherent randomness**. It does not matter how well the process and the initial (starting) conditions are known, certainty is limited to what the outcome will be. Although it may be inherent to nature, the randomness can often be quantified with probabilistic models.
- **Natural variation**. As for nature, the natural systems change in time and place, also the parameters of interest will change. Therefore, despite the measurements, there is always uncertainty about natural conditions. Although it requires some careful consideration the variation can also be quantified. Consideration should include the possible range and relative probabilities of the unknown quantities.
- **Measurement error**. Measurement error causes uncertainty about the value of the measured quantity. The measurement error can be estimated by statistical methods if several samples are taken. If the extent of the measurement error can be estimated, it can be relatively easily dealt with in probabilistic models.
- **Systematic error** in the measurements results from a bias in the sampling and is more difficult to quantify, or even notice. If the systematic error goes unnoticed, it may have cumulative effects on the models that are built on the data.
- **Model uncertainty**. Models are always abstractions of the natural system. Some less important variables and interactions are left out, and the shapes of the functions are always abstractions of the real processes. One may have insufficient knowledge about the relevant processes, the shapes of the functions and their parameter values. Uncertainty of the model parameters can be accounted for in probabilistic models much the same way as natural variation, with careful consideration of the range of possible values and their probabilities; while uncertainty about the model's structure, i.e., uncertainty about the cause-and-effect relationships, is often difficult to quantify.
- **Subjective judgement-based uncertainty** occurs due to the interpretation of data, especially when the data are scarce or error-prone.

In addition to previous uncertainties, there is linguistic uncertainty, that comes with language issues, but this type of uncertainty is considered unnecessary in the context of bridge management.

Aleatory uncertainty cannot be reduced, but epistemic uncertainty can be reduced by improving the knowledge or making deterioration models more accurate (A.-S. Ang & Leon, 2005) (Goulet et al., 2015). Moreover, aleatory uncertainty may become epistemic once the structure is constructed (Faber, 2000) (Goulet et al., 2015). The distinction between these two uncertainties is frequently determined by modelling choices (Kiureghian & Ditlevsen, 2009), but conclusively it is impossible to distinguish or separate various types of uncertainties (Uusitalo et al., 2015). Deterioration models can be overly sensitive to change of the parameters of the input random variables, and when the modeller is aware of the various sources of uncertainty, advanced inspection methods like monitoring may provide a piece of useful information to improve the accuracy of models by reducing the level of epistemic uncertainty (Frangopol, 2011b). For example, based on Biondini and Frangopol (Biondini & Frangopol, 2015) the effect of monitoring on the life-cycle performance prediction is qualitatively shown in Figure 5 for both underestimation Figure 5 (a) and overestimation Figure 5 (b) of service life.

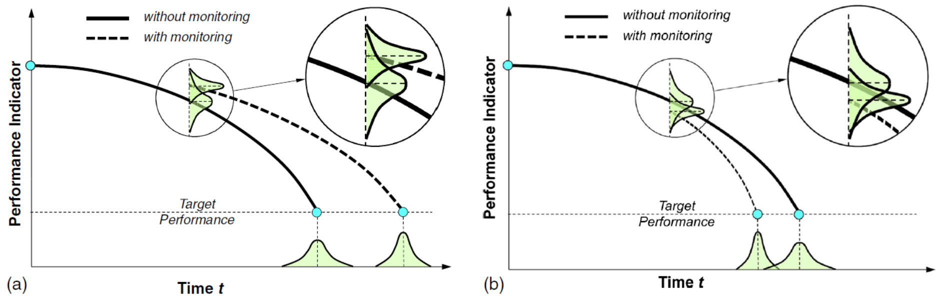

Figure 5. Updating the performance profile based on the results of monitoring: without monitoring service life is (a) underestimated or (b) overestimated. Adapted from Biondini and Frangopol (2016).

Overall the selection of optimal inspection or intervention strategies requires consideration not only of uncertainties related to degradation phenomena, but also the quality of inspections among others (Frangopol, 2011b). Inspection results are characterized by uncertainty related to damage detection, and the uncertain nature of the function leads to the fact that it is not always possible to determine accurately the physical condition of the structure (Valdez-Flores & Feldman, 1989, Madanat, 1993). As a result, inspections may fail to identify if there is a real need for an intervention or the extent of the required maintenance. To evaluate the accuracy of inspections, Sanchez-Silva et al. (2016) concluded, that there are two different approaches. The approaches can be distinguished by good and wrong assessment where the probability of good assessment determines the probability of detecting an event (crack, defect, concentration, etc.) that exists in reality, and probability of wrong assessment establishes the probability of detecting an event that does not exist. In the wrong assessment, two types of errors can be distinguished (Sanchez-Silva and Padgett, 2016):

- The structure is in a good state (operating above the minimum threshold level) but is judged to be bad and it is repaired.
- The structure is in a bad state (e.g., failure state) but is judged to be good and it is not repaired.

To model such uncertainty as a function of the damage level, taking into account probabilities of damage detection, different methods have been proposed including false alarms by Faber and Soerensen (2002) or Sheils et al. (2012), correct or incorrect assessments after inspection by Orcesi and Frangopol (2011).

To conclude, uncertainty is unavoidable in bridge management and despite the conceptual difficulties in the quantification of variables and making predictions, an appropriate understanding of the existence of uncertainties will help decision-makers to benefit from the potential outcome.

1.3 Performance assessment

Performance assessment, as a part of data collection, is mostly done with visual inspections, which aim at evaluating the condition state of infrastructure, through the identification and classification of defects and anomalies that affect its performance, considering its intensity and extension. The inspection record has a fundamental role for the knowledge of the condition state of the infrastructure and to determine an optimised maintenance plan, allowing prioritizing the maintenance and/or rehabilitation actions for the infrastructures with higher deterioration levels. Maintenance action records are essential for the manager to be informed about the nature of all maintenance actions that were carried out and of the costs of the conservation works of any infrastructure (Ryall, 2001). The condition rating or similar (condition rating, condition index, condition state, damage index etc.) is adopted to describe the condition state of the existing structure and is not a direct measure of structure safety but rather a measure of the severity of observable defects and are usually based on a discrete scale based on objective and uniform criteria. It should indicate the overall condition of the structure and is mostly represented by an arbitrary scale with linguistic terms like high, moderate or low. The scale of condition ratings is not universal, usually, each management system develops its rating scale (Elbehairy, 2007). Based on the review of Frangopol (2011), the main disadvantage of qualitative indicators is that the actual structural condition and safety level are not explicitly or adequately accounted for and may fail to account for actual structural performance. However, any attempt to standardize the criteria is efficient because it allows the information to be treated invariably for the whole network covered by the management system and in some way reduces the variability of the process of evaluating the performance of the infrastructures. COST Action TU1406 aimed to standardize the establishment of quality control plans including condition assessment for roadway bridges (Matos et al., 2016) and visualised the current situation of different scales used for structural inspection (Figure 6).

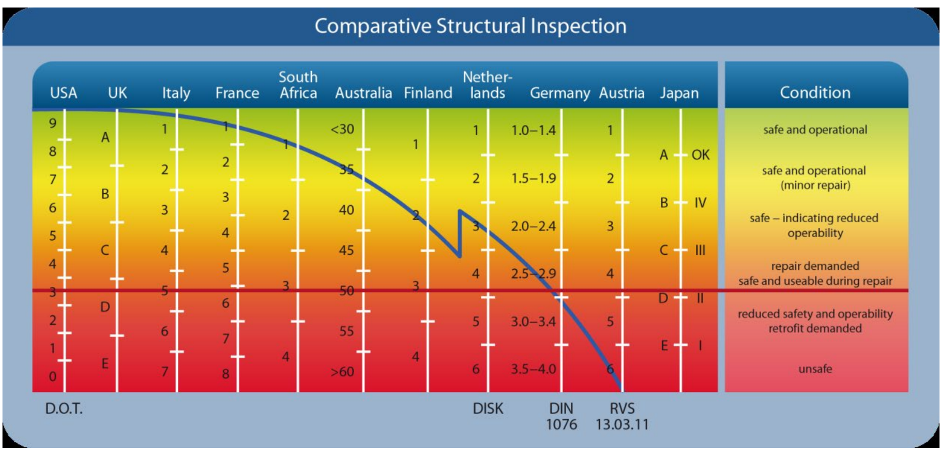

Figure 6. Comparison of structural inspection for performance over time (Wenzel & Pakrashi, 2019).

Hajdin et al. (2018) stated that the condition state is a vague measure to describe the deviation of the inspected bridge from the "as new" condition. Moreover, the maintenance policies are also often defined based on periodic observations of the structural condition and even if a more expensive in-depth investigation is triggered, the intervention action will highly likely follow. Overall, the qualitative condition assessment is widely used in the management of bridges (Golabi & Shepard, 1997) (Mirzaei et al., 2014).

In addition to visual inspection based qualitative indicators, there are also quantitative performance indicators, where numerical values are coupled to the observed phenomenon by counting or measuring (Hajdin et al., 2018). Quantitative performance indicators normally rely on technical design principles and rules or models for structures, like described in currently available Eurocodes or the fib Model Code 2010. These principles offer mathematical approaches to ensure that time-independent effects on a structure and resistances of material are under the expected reliability of a structure during the chosen reference period (Zimmert et al., 2020). Quantitative approaches are generally well-founded in terms of scientific research outcome and the likelihood of unexpected change in these assumptions during a construction's service life is low. These quantitative reliability assessments vary in sophistication and can be divided into two main groups – measurement and model-based assessments, components are usually related to measurement or acquisition of load effects and resistance, calculation of load effects on structural models and verification of serviceability and safety (Rücker et al., 2006). If the resistance models of a structure are statistically quantitative, then possible time-dependent effects on a structure, like for example traffic- or environmental loads, are on the other hand constrained statistically in terms of a future prediction. Time-dependent material degradation processes are considered only in an implicit mathematical way, for example by defining minimal concrete cover rates with respect to expected environmental and or mechanical exposure. However these aforementioned time dependent processes are in many cases of a not fully predictable, or mathematically describable nature and may in addition change during a structure's service life time (Helland, 2013). In comparison to qualitative assessment, expert judgement as the source of uncertainty estimates can easily be criticized as subjective. However, lacking data to estimate the variances by, the experts who have devoted their careers to studying these questions might be better sources of information than any hasty quantitative models not directly made for the purpose. For expert judgement, various methods exist to help and support the experts in the evaluation task. However, the facilitator also must be careful to make sure that the experts in fact evaluate the desired quantity, not something related but distinct (O'Hagan et al., 2006).

The main difference from the management point of view between visual inspection and reliability-based performance assessment is that the first considered cost-efficient and very valuable source of information, but reliability assessment of safety and serviceability is regarded as not time and cost-efficient because it involves additional non-destructive material investigations or load testing and structural analysis (Hajdin et al., 2018). As a result, some authorities in Europe, including Estonia, use only condition rating for maintenance planning and asset management. In most cases, they do not use the obtained data as input into load and resistance assessment. From the other point, there are already owners who use advanced non-destructive methods in inspection and condition assessment (Paulsson et al., 2010).

Based on the Sustainable Bridges project report (Paulsson et al., 2010) some highway agencies have issued guidance about the use of non-destructive testing (NDT) for reinforced concrete bridges and during the project a multi-level flowchart of possible assessment phases were also developed (Figure 7). However, no standards for use in bridge assessment procedures exist in most European countries. Data processing and presentation is often insufficient and not interpretable for the authorities. In many cases, specialised laboratories can do only feasibility studies with more refined inspections, which are necessary in case of heavy safety problems.

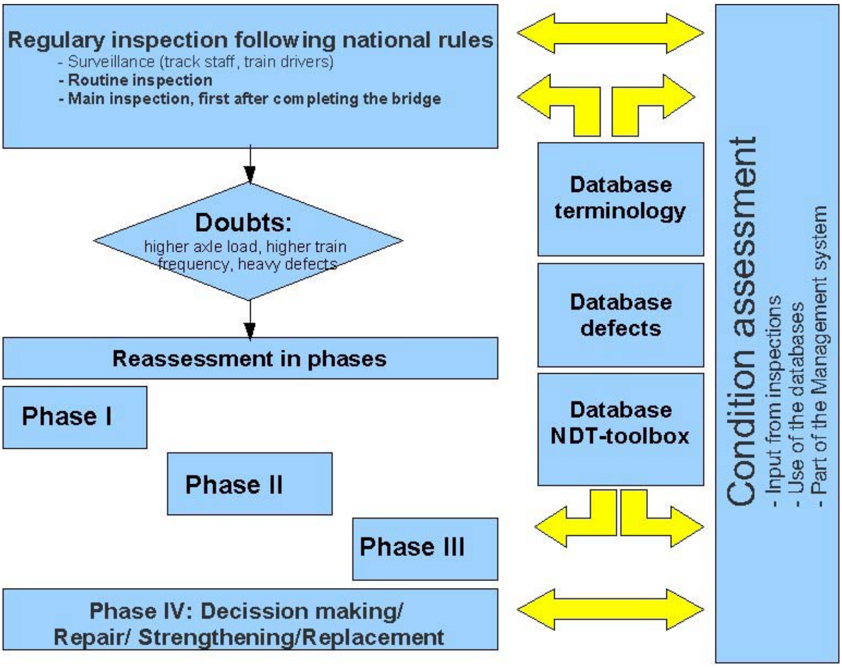

Figure 7. Sustainable Bridges project flowchart showing how regular inspection is linked to condition assessment and structural performance assessment (Reassessment) (Helmerich et al., 2007).

After the Sustainable Bridges project, a collaborative project to improve rail transport infrastructure called the Mainline Project (Elfgren et al., 2015), where it was stated that the level of assessment accuracy needs to be commensurate to the level of material/structure accuracy, i.e. there is little point in doing an advanced assessment if material properties etc. are only known to the nearest 10% or similar. There is also the issue of old materials not being to current Eurocode specifications (e.g. steel reinforcement anchorage/bond in concrete – this means that there may be limits to the use of plasticity/rotational capacity). This means that some form of monitoring or intrusive work, and incredibly detailed inspection of the structure, may need to be considered before undertaking advanced assessment methods. Wisniewski et al (2012) came to a similar conclusion and stated that bridges that fail to pass initial safety checks should be re-evaluated using Intermediate Level analysis procedures, which would involve any combination of the following. Additional more thorough inspections with possible field testing for material properties to obtain better estimates of member strengths. This is also an issue considered in this deliverable, where the information of "in situ" or material testing can be introduced in the assessment process employing Bayesian updating (Elfgren et al., 2015). During the project, a list of potential inspection and assessment methods could increase the reliability of data and that would give a piece of better knowledge about the condition of a structure. They also stated that the negative side of the advanced method would be, that the needed apparatus may be expensive to use.

Zimmert et al. (2020) stated that the effort and costs of this stepwise strategy can become very high with dependency on the number of boundary conditions surveyed and data to be collected and evaluated. The question of international standards should give deeper proposals for the usage of this type of service life verification and intervention strategy or more practical and economical feasible advice should be given. This possible option is the execution of a condition-based intervention strategy, for example, proposed in fib Model Code 2010 (FIB et al., 2013). Applying a condition-based intervention strategy on a construction means, that the actual state of the structure itself is examined and compared to the reference state (FIB et al., 2013). As soon as predefined forms of damage occur, the decision-making processes for intervention can be performed. Condition-based conservation strategies and, in detail, periodic inspections of structures for further decision making are yet executed in different countries in Europe and national standards were developed (Zimmert et al., 2020). If these standards and techniques can be implemented into the fib Model Code, a decision-making guideline for periodic inspection intervals according to a structures requirement of safety could be also used. This guideline introduced by Zimmert et al. (2020) refers to predefined classifications of structures or structural parts into Consequences Classes and Robustness Classes, for which it is proposed to deliver examples for classification.

The assessment is typically carried out during a short period, but deterioration and damages occur for a long time. Therefore the accurate performance assessment despite the modelled variable needs predictive modelling (Elfgren et al., 2015).

1.4 Deterioration models

Deterioration models are considered a critical component of a management system. This component has the function of simulating the degradation process of the infrastructure. However, the results provided by the deterioration models are subjective, because they are usually associated with a significant level of uncertainty. To capture the variability of the degradation process, stochastic deterioration models are used to predict the future performance of the structures. The majority of deterioration models is based on Markov chains (Zambon et al., 2018). There are also a considerable number of studies based on reliability-based approaches and, recently, the Petri net formalism has been used to model the deterioration process (C. A. R. Ferreira, 2018).

Deterioration models aim at predicting the degradation process of the infrastructures. Its purpose is to assist the manager in making decisions about the actions to be performed on the structure. In general, deterioration models can be based on inspection results, estimates obtained through expert opinion or by combining these two methodologies (Kallen, 2007). Deterioration can also be defined as the process of decline in the condition of the infrastructure resulting from normal operating conditions, excluding damage from extreme events like earthquakes, accidents, or fires (Elbehairy, 2007). Due to constant interaction with the environment, civil engineering assets are exposed to different types of actions, including environmental stressors and loads, which directly or indirectly contribute to their deterioration over time.

The common way to build a quantitative environmental model is to describe the relationships between model variables using mathematical equations with deterministic values (Jackson et al., 2000). These values are found either in literature, by fitting equations to data, or, if no such information is available, through iterative search in which the model outputs are compared to observed system behaviour (Jackson et al., 2000). However, it is not always an easy task to identify how these physical changes lead to a reduction in system capacity, which is how we define degradation. Modelling can clarify our understanding of human nature interactions, point out where the largest gaps in our knowledge lie and distinguish between competing hypotheses.

Because of the challenges in modelling a variety of physical changes that cause system performance to degrade over time, most degradation modelling asserts two primary degradation classes, namely (Liu & Frangopol, 2004) (Petcherdchoo et al., 2008) (Frangopol et al., 2009) (Ghosn et al., 2016):

- Continuous (progressive or graceful) degradation.
- Degradation due to discrete occurrences (shocks).

For a variety of reasons, it is conceptually important to classify degradation in this way. From an observational viewpoint, certain mechanisms, such as corrosion or continuous material removal due to friction or heat, fit naturally within the progressive deterioration category. These mechanisms generally involve very small changes in physical properties that occur continuously over a long timescale. Other changes, such as loss of material due to a sudden collision and disruptions due to failure of a component that may not cause immediate system failure, are more appropriately viewed as shock degradation. To model the continuous degradation process three main approaches can be used:

- Reliability-based models.
- Condition-based models.
- Damage process-based models.

None of these approaches has shown evidence of being able to be applied generically, because all methodologies have their advantages and disadvantages. Reliability-based models deal with the reliability index, they can quantify physical parameters such as material properties, stress conditions, structural behaviour, among others in a probabilistic framework. The condition-based model is more suitable to incorporate information from visual inspections, but it cannot be used to assess the reliability of a structure in terms of strengths and stresses. Damage process-based models are mostly analytical and used when quantification of a specific environmental-related process like chloride ingress or carbonation is needed. These models are widely investigated but have not found application in bridge management. Besides, bridges normally consist of many components that have several failure modes and different consequences of failure (Lounis & Madanat, 2012) (Frangopol et al., 2004) and covering all aspects may be cumbersome. Since it is easier to evaluate visually superficial defects than internal (Ellingwood, 2005) processes, the performance assessment of bridges relies on visual inspections. The condition-based models, especially as Markov chains, are still today the most used approach in deterioration models (C. A. R. Ferreira, 2018) and are also introduced more thoroughly in Paper IV.

1.4.1 Markov chains

The Markov chains are stochastic processes used extensively for modelling the deterioration in different fields of civil engineering and existing bridges (Butt et al., 1987) (Thompson et al., 1998) (Consultant & Johnson, 2005) (Sánchez-Silva & Klutke, 2016).

The first deterioration models based on the Markov process were developed by Golabi (1983) to describe pavement condition changes over time. The use of Markov chains in bridge management started in the early 1990s when Cesare et al. (1992) applied discrete Markov models to evaluate the deterioration of highway bridges. The study showed that a correlation between deterioration of elements existed and it is possible to apply these processes for the prediction of average condition considering the effect of annual repairs.

In 1994 Scherer and Glagola investigated the applicability of Markov processes focusing on the compliance of Markovian property and state-space explosion (Scherer & Glagola, 1994). Regarding verifying the compliance, a frequency analysis of two possible sequence occurrences with the same present and future states, but the different past states was employed. The inference analysis using chi-square statistic was used and the results revealed that the property is a good assumption for bridge deterioration models. To reduce the state-space a classification of bridges into groups with similar performance characteristics were found useful.

In 2006 Morcous investigated the use of Markov chain models in predicting the deterioration in bridge management and came to five important conclusions (Morcous, 2006):

- Markov models can reflect the uncertainty from different sources like initial condition, presence of condition assessment errors and the inherent uncertainty of the deterioration process.
- The models account for the present condition to predict the future condition.
- The efficiency and simplicity of the models allow for the management of bigger networks.
- Commonly the models use discrete parameter Markov chains, to eliminate the computational complexity and simplify the decision-making process. Unfortunately, the inspections are carried out with a variation, which results in condition data that are not equally spaced and cannot be easily utilized in developing and updating new models. The evaluation of variation was investigated using the Bayes rule and results showed that the variation in the inspection period may result in a 22% estimation error.
- Most models use first-order Markov chains that assumes state independent approach where the future condition depends only on its present condition. This assumption was also used to simplify the deterioration prediction even though deterioration is a nonstationary process, which means that the time elapsed in the initial condition state affects the probability of transition to the following state. Investigations showed that the state independence assumption is acceptable for the network-level analysis with 95% of confidence.

Based on the developments and investigations, Markov chains are considered a simple way to predict the future condition state of elements over time in situations where the full history of the elements is not available (Kalbfleisch & Lawless, 1985) (Morcous, 2006). This assumption allows overcoming the problem of lack of records in the civil infrastructure because infrastructure, usually, is not continually monitored (Kallen & Van Noortwijk, 2006).

Mathematically the process describes the transitions between finite state space and satisfies the Markovian property. The Markovian property states that the future state only depends on the present state, not considering the process up to the present state (Sánchez-Silva & Klutke, 2016). The Markov property for a discrete-time process is satisfied if (Eq. 1):

$$P(X(t+1)=j|X(t)=i, X(t-1)=i_{t-1}, \dots, X(1)=i_1, X(0)=i_0) = P(X(t+1)=j|X(t)=i), \tag{1}$$

where P represents probability distribution, $(X(t), t = 0,1,2, \dots)$ is a stochastic process that takes values in a countable state space, s, $(t = 0,1,2, \dots)$ is the index set represents time and $X(t)$ refers to the condition state of the process at time t. These models are based on a discrete scale, where the transition between states is defined (Eq. 2):

$$\begin{bmatrix} C_1 \\ C_2 \\ \vdots \\ C_i \end{bmatrix}_{t+\Delta t}^{T} = \begin{bmatrix} C_1 \\ C_2 \\ \vdots \\ C_i \end{bmatrix}_{t}^{T} \times \begin{bmatrix} p_{11} & p_{12} & \cdots & p_{1j} \\ 0 & p_{22} & \cdots & p_{2j} \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & 0 & p_{ij} \end{bmatrix}_{\Delta t}, \tag{2}$$

where $C_{t+\Delta t}$and C_t are condition vectors at time $t + \Delta t$ and t, respectively. Vectors are defined as the probability of an element being in each performance state, C_j. The probability of transition between state i and j from instant t and $t + \Delta t$ is defined by p_{ij}, which is an element of a matrix P. Considering that the elements belonging to the lower triangle are null, is equivalent to say that the infrastructure deteriorates naturally without improving their condition state. The element p_{ij} takes the value of 1, and represents the absorbent state of the performance scale. Once an asset reaches this condition, it remains in that condition state forever unless a maintenance action is carried, and its condition state is improved (Sánchez-Silva and Klutke, 2016). Although in a discrete Markov process, the probability $P_{\Delta t}^{final}$ is formulated for a constant time interval Δt, it is possible to obtain transition probability matrices for time intervals greater by geometric progression with exponent k, as stated (Eq. 3):

$$P_{\Delta t}^{final} = (P_{\Delta t})^k, \tag{3}$$

Since the discrete Markov chains had simplifications, Kallen and van Noortwijk (2006) applied the continuous-time Markov process to model the uncertain bridge condition over time, which on the contrary allows transitions to occur in a continuous timescale. The authors prove that difference in complexity is small when focusing on the transition probability matrix P. The main difference from the discrete Markov process is that the continuous process is defined with a transition intensity matrix Q (Eq. 4):

$$\frac{\partial}{\partial t} P\,(t) = P(t) \times Q, \tag{4}$$

where $\frac{\partial}{\partial t} P\,(t)$ is the probability matrix at time t, $P(t)$ is the initial probability distribution and Q is the intensity matrix. The paper from Kallen and van Noortwijk (2006) focused on the statistical estimation of parameters in four types of transition, looking for the best fit between optimised model estimation and observed condition. The optimal transition was based on the concept of maximum likelihood described by Kalbfleissch and Lawless (1985). A similar approach has been used in many former kinds of research and applications (C. Ferreira et al., 2014) (Denysiuk et al., 2016) (Hamida & Goulet, 2020). Overall, if the time between inspections is not discrete then the continuous-time Markov chain should be used instead of discrete (Kallen & Van Noortwijk, 2006). Mathematically the continuous process $\{X(t), t \geq 0\}$, is expressed as follows (Eq. 5):

$$\begin{aligned} P(X(t + \Delta t) = j | X(t) = i, X(u) = x(u), u < t) = \\ P(X(t + \Delta t) = j | X(t) = i), \end{aligned} \tag{5}$$

where, i, j and $x(u)$ are values of space s and $t, \Delta t$ represent time greater than 0. It is assumed that, in transitions between condition states, the length of time spent in condition state i before marking a transition is an exponentially distributed random variable with an additional parameter that depends only on condition state i. The next state depends only on the current state etc. Kallen and Noortwjik (2006) proposed to use the transition intensity matrix in case of using continuous time-independent and state-dependent Markov process transition intensity matrix through the Chapman-Kolmogorov equation (Karush, 1961) as follows (Eq. 6):

$$P = e^{Q\times\Delta t} = \sum_{n=0}^{\infty}\frac{(Q\times\Delta t)^n}{n!}, \tag{6}$$

where P is the transition matrix and Q is the intensity matrix, which represents the instantaneous probability of transition between the state i and j, where $j \neq i$.

The intensity matrix for the deterioration process is calculated using the state-dependent and time-independent model (Kallen & Van Noortwijk, 2006) (Eq. 7):

$$Q = \begin{bmatrix} -\theta_1 & \theta_1 & 0 & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & -\theta_i & \theta_i \\ 0 & 0 & 0 & 0 \end{bmatrix}, \tag{7}$$

where θ_i is the instantaneous transition probability between adjacent state i and j. The initial estimate of matrix Q is calculated through (Eq. 8):

$$\theta_i = q_{ij} = \frac{n_{ij}}{\sum \Delta t_i}, \tag{8}$$

where n_{ij} is the number of elements that moved from state i to state j, and $\sum \Delta t_i$ is the sum of intervals between observations.

Ferreira (2018) concludes that the main advantage of the Markov process is its simplicity, the use of exponential distribution to describe the transition between condition state and the existence of analytical expressions for the probability distribution.

1.4.2 Alternative deterioration models

Although Markov processes are mostly used in modelling condition-based deterioration several alternative methods using a linear approach, Bayesian network, Artificial Neutral Network or analytical damage-related models can be considered.

For example, Neves and Frangopol (2005) proposed a linear deterioration model that integrates both condition and safety indicators, producing a more consistent measure of the effect of deterioration on serviceability and safety of existing structures. The condition profiles under no maintenance are defined using three random variables: initial condition, C_0, time of initiation of deterioration of the condition, t_{ic}, and deterioration rate of a condition, a_c. The time-dependent deterioration profile of the condition is considered linear as follows (Eq. 9):

$$C(t) = \begin{cases} C_o & 0 \le t \le t_{ic} \\ C_0 - \alpha_c(t - t_{ic}), & t > t_{ic} \end{cases}, \tag{9}$$

where $C(t)$ is time-dependent condition profile, and t is time. A similar formulation was described by Frangopol et al. (2004), but instead of a condition profile, a time-dependent reliability index was proposed to address the need for a quantitative approach.

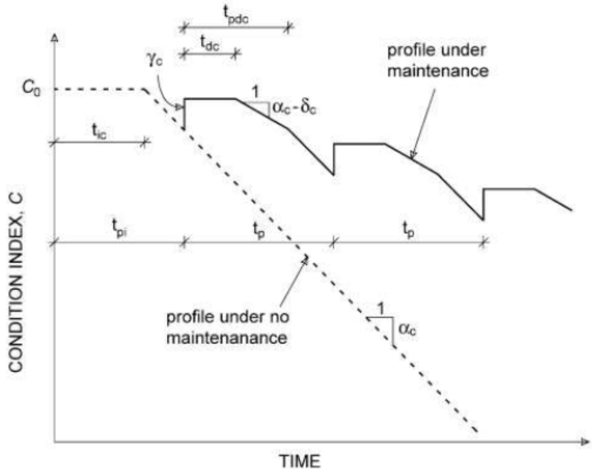

Figure 8. Condition index profiles under no maintenance and maintenance (Neves and Frangopol, 2005).

To overcome the issues of past information and stationary transition probabilities of condition states several models have been developed. For example, Wang et al. (2012) proposed a model using Dynamic Bayesian Networks that uses Bayesian updating to utilise different types of data in condition prediction. Similar Bayesian Network-based deterioration models have been also proposed by Rafiq et al. (2015) and Torre et al. (2017), who combined the modelling with Markov chains. Also, Artificial-Intelligence based models are used to generate missing information artificially for better transition estimation. For example, Lee et al. (2008) proposed Backward Prediction Model based on Artificial Neutral Network that employed non-bridge factors like traffic, climate and population information, to predict the historical bridge condition ratings (Figure 9).

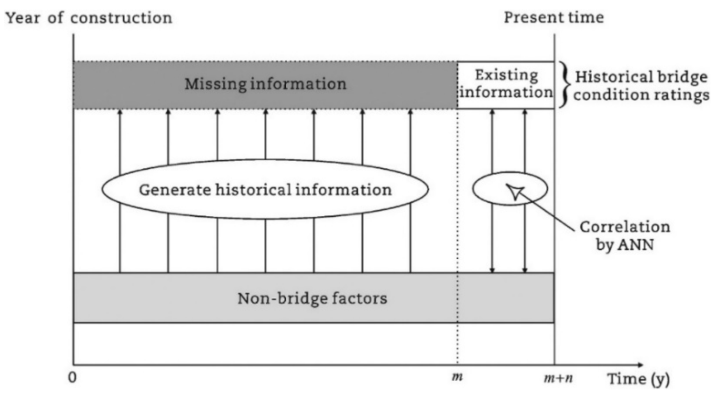

Figure 9. Outline of Backward Prediction Model (Lee et al., 2008).

More complex Artificial Neutral Network computational models have been proposed, but since the approach can be used only to generate missing information then additional utility functions are needed (Srikanth & Arockiasamy, 2020).

The damage based deterioration models are used to model quantitative physical parameters and similarly to stochastic models, many research studies have been carried out to model the deterioration of concrete structures in the past decades. For example, Roberts et al. (2000) proposed an empirical corrosion model for reinforced concrete taking into account the variation of corrosion and delamination. Also, The International Federation for Structural Concrete (FIB) has developed several documents like Bulletin 34: Model code for Service Life Design (béton, 2006) dealing with various deterioration mechanisms including carbonation and chloride ingress. These models are costly in terms of data collection for large networks and have not been implemented in practice.

Zambon et al. (2019) proposed a combination of carbonation-induced corrosion and stochastic models to overcome the limitations of both models. A relationship between different deterioration phenomena was presented using the description of condition states and period between different states (Figure 10).

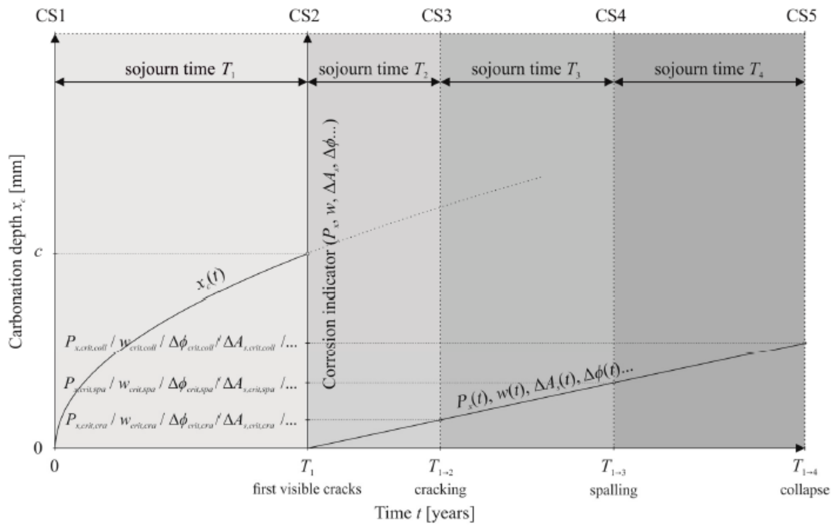

Figure 10. Relationship between periods of the carbonation-induced corrosion and sojourn times of condition states (Zambon et al., 2019).

The most recent studied bridge deterioration models are related to the use of Petri Nets models. This approach allows modelling individual elements of a bridge by considering the other elements and maintenance activities. For example, Le and Andrews (2016) used the Petri net approach to develop a bridge management model. The bridge model was formed from sub-models of bridge elements considering the deterioration process, the interaction and dependency between different elements, deterioration processes, inspection, and intervention activities. The deterioration process itself is governed by Weibull distributions, with parameters obtained from the historical records.

1.5 Decision-making

The application of life-cycle optimal design and management concepts in selecting the materials and structural attributes can play a significant role in maximising the performance and minimising the total life-cycle cost associated with several cost components, including the initial construction cost and the costs of operation, maintenance, inspection, monitoring, repair, and demolition, as well as the indirect costs of non-performance or failure (Akiyama et al., 2019).

Decision-making is also a continuous selection process of the best outcomes that affect the overall performance of a structure and starts already in the planning stage. The initial decision influences the service life, deterioration, and conservation strategies. Zimmert et al. (2020) presented a framework of decision-making options in different life-cycle stages of a structure (Figure 11) and stated that decision-making related to existing structures is significantly more complex, demanding further studies and the execution of a probabilistic approach. Despite the decisions, it is important to regularly survey the condition according to the assumptions made during the design stage.

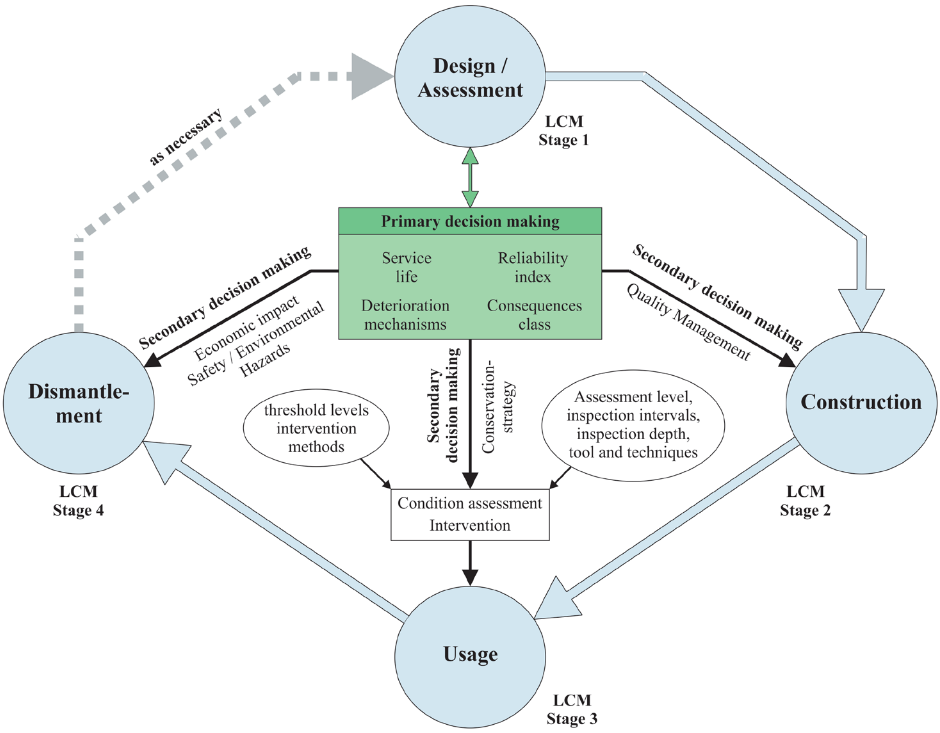

Figure 11. Decision-making steps through the life-cycle of a structure (Zimmert et al., 2020).

The probabilistic, integrative decision support models can derive their data from three types of sources: first-hand data, expert knowledge, or pre-existing (probabilistic or deterministic) models (Uusitalo et al., 2015). The experts' task may be eased by investigating areas or cases that are deemed sufficiently similar, and upon which data exists. For example, values from similar structures built in different locations may be useful here. The range of relevant observed values can give information about the plausible range of values this variable can get, and therefore may also indicate how large uncertainty is associated with the prediction of the deterministic model. Every model is just "a stylized representation or a generalized description used in analysing or explaining something" (Hilborn & Mangel, 1997). Thus, how much effort and resources should be put into translating a deterministic model to probabilistic form is a case-specific question and a cost-effective solution for decision-making might be combined approach because commonly decisions are based on multiple and conflicting criteria that are subject to different levels and types of uncertainty. Normally the process is starting with only light prior assumptions about the qualitative causalities in the model. With the simpler model version, the decision modeller could then analyse by, for example, conducting a value of information analysis (Clemen & Winkler, 1985), what elements in the model are mostly affecting the ranking order of the analysed decisions.

Commonly multi-criteria decision-making (MCDM) is used, because of the ability to systematically combine different inputs with outcomes and help to rank the alternatives. Frangopol and Liu (2007a) made the first overall review of the recent developments considering multiple criteria in decision-making of bridge management and Kabir et al. (2014) made a comprehensive review of different MCDM methods used in infrastructure management. The main MCDM approaches based on Kabir et al. (2014) are listed below:

- Weighted sum model.
- Weighted product model.
- Compromise programming.
- Analytical hierarchy process (AHP).
- ELECTRE (translated from French ELimination Et Choice Translating REality).
- TOPSIS (Technique for Order of Preference by Similarity to Ideal Solution).
- PROMETHEE (Preference Ranking Organization METHod for Enrichment of Evaluations).
- VIKOR (translated from Serbian Multicriteria Optimisation and Compromise Solution).

The above MCDM approaches share some common mathematical elements like values for alternatives are assigned for each criterion, and then multiplied by corresponding weights and finally combined to produce a total score (Huang et al., 2011). Decision-making can support in many fields, for example, in Chapter 3.1 AHP is used to rank criterions necessary for evaluating suitable NDT methods.

Although the decision-making models are widely used in the research field, common decision-making relies highly on the experience of engineers, which means that there is a knowledge gap in how data is utilised to make decisions (Woldesenbet, 2014). Wu et al. (2020) reviewed journal articles and identified four challenges related to data-driven decision-making based on researches:

- Poor definition of data needs.
- Lack of method to assess data quality.
- Lack of data integration.
- Inadequate consideration of operational issues.

This list gives a good overview of the current situation in decision-making, meaning that only a few problems can be solved using quantification of the uncertainties related to the quality of performance assessment and integration of visual assessment and NDTs.

2 Bridge management of Estonian bridge network

Although the framework applies to any management system, Estonian national road bridge data was used. Overall, there are approximately 4200 roads-, railway and pedestrian bridges in Estonia. A bridge is defined as a structure with a span of over three m and built to cross an obstacle (Minister of Economic Affairs and Infrastructure, 2018). Structures are owned by a different institution and the biggest owners are the Estonian Transport Administration (ETA), Estonian Railways LTD., State Forest Management Centre, and local authorities. Most of the owners do not have any management system implemented which means that they manage the structures without any systematic approach.

The most systematic approach is implemented by ETA, where the condition data have been uniformly collected since 2005, but any advanced frameworks have not been used to analyse the data. For a better understanding of Estonian national bridge management, policies, performance assessment and decision-making are introduced.

2.1 Current bridge management regulation in Estonia

The bridge management is governed by Building Code (*Building Code – Riigi Teataja*, 2021), Road Traffic Act (*Road Traffic Act – Riigi Teataja*, 2021) or by other Acts related to the built environment. Main requirements are applied according to Chapter 11 of Building Code, wherein *§ 97. Road maintenance* is stated, *"Roads and any civil engineering works necessary for the functioning of the road must be maintained such that they conform to the requirements and such that the conditions for safe traffic are ensured"*.

The more specified regulation Act *"Requirements for the condition of roads"* (Minister of Economic Affairs and Infrastructure, 2018), covers condition state requirements for roads and bridges and wherein § 32. Inspection of bridge condition is stated that condition should be inspected regularly by maintenance operator and once in every three years during the general inspection. The time, activities and data requirements are determined by the owner.

Activities of ETA are financed under a road management plan applied according to *§ 1^1. Financing of road management and road management plan,* where is stated that national road management is prepared for four years and updated annually. Bridge management is also covered within the management plan, where is stated that the financial volume is based on the condition analysis of bridges (ETA, 2020) and data is collected during a more thorough general inspection, which is carried out once in four years. The development of the currently used condition assessment method started in 2003 and was done by consultant company *Teede Tehnokeskus AS* using program Pontis, which had a database containing bridge condition data, traffic needs, accident data, maintenance, improvement and replacement costs, available money, etc. Not all of the available possibilities were used, but it was successfully in use until 2013 when ETA decided to take bridge management under its responsibility. In the next chapters, the performance assessment and intervention analysis are introduced in more detail.

2.2 Performance assessment

The current performance assessment of national bridges relies mainly on visual inspections that can be divided into regular and general inspections, but data is collected only from the latter. During the general inspection condition state of each element is numerically evaluated describing the severity and area of the damages. Also, damages with urgent intervention needs are noted and photos of elements and damages are taken.

There are no qualification requirements for inspectors and only simple manuals with photos or overall description of damages are available. Although proper documentation with instructions is missing, all elements are systematically numbered, and it is mandatory to follow correct numbering. Defects are described without classification. The assessment relies on condition states of elements, which are divided into 4 different states that can be described in Table 1. During the inspections, it is possible to add comments in case of condition state is lower than 1.

Table 1. Description of different condition states. From Paper IV.

Condition state	**Description**	**Intervention activity**
1 - Very good	Element has no remarkable defects or wearing marks. Overall appearance is good as new and only small damages can occur like bleaching.	Regular maintenance
2 - Good	Element has minor superficial damages, wearing and deterioration processes can occur. Overall appearance is the clean and small deviation of deterioration processes are allowed. Minor repair works are needed.	Local repairs
3 – Poor	Element has defects, like corrosion, but the severity of the damage is not affecting functional requirements. Overall appearance gives a clear indication, that deterioration processes are damaging the element. Repair is needed.	Repair
4 – Very poor	Element has defects, that could affect overall or element performance	Replacement or reconstruction

The condition states are evaluated without a time limit, which means that the only information is conditional and the intervention method is only included as a suggestion. Since 2015 extra letters are used to add the time limit for potential activity, Letter A refers to immediate intervention and letter B for intervention within five years. The used time limit is connected to element defects that would affect the safety (both structural and traffic) of construction. The example of collected inspection data in table form is presented in Table 2.

Table 2. Example of data collected during general inspection (Sein & Rentik, 2017).

Element			**Ins. date**	**Condition state**					**Comment**
Name	Number	Amount	10.05.2007	1	2	3	4	Flag	
Concrete beam	100	15 m		0	11	4	0	B	Rust in the ends
Elastomeric bearing	312	8 pcs		8	0	0	0		

One element is divided into countable increments and every increment is then evaluated into a specific condition state. The overall condition state CS_i of an element is calculated as a weighted average based on the overall quantity of units and state factors (Eq. 10):

$$CS_i = \left(1 \cdot CS1_i + \frac{2}{3} \cdot CS2_i + \frac{1}{3} \cdot CS3_i + 0 \cdot CS4_i\right), \tag{10}$$

where CS_i is an overall condition state of an element, $S1_i$… $S4_i$ are values taken from the inspection data and multiplied accordingly to average state indicators. The overall condition state of an element is just an intermediate value that is used in the calculation of bridge condition.

The condition of a bridge is presented also numerically with Condition Index (CI), which is calculated similarly as in most of the DOTs in the US. The result is expressed with one number between 0-100 and is calculated as a weighted average of condition states of elements (Eq. 11):

$$CI = \frac{\sum(WF_i \cdot CS_i)}{\sum WF_i}, i = 1 \ldots N, \tag{11}$$

where CI is the number that reflects the condition of a bridge, WF_i is the weight factor showing the importance of the element (Table 3) and CS_i is an overall condition state of an element. Weight factors were revised in 2015 to make the hierarchy of elements simple and clear. During the implementation phase, the weight factors were connected to element replacement costs, but since the prices were based on 17 bridge values, then the weight factors were between 29.9 to 164892.1, which meant that the difference of least and most important element was 5512.9 times.

Table 3. Distribution of different weight factors according to the element group (Sein & Rentik, 2017).

Weight factor	Element group
3	Abutments, piers, crossbeams, beams, girders
2	Waterproofing, bearings, deformation joints, deck slab
1	Road surface, barriers, slopes, drainage

From the BMS view, the outcome of the general inspection of a bridge is just one number, CI, which reflects the need for intervention. It is agreed in ETA, that an optimal level is reached when the bridge will be repaired before the CI reaches 70 and with CI less than 33, the closing of the structure should be considered. Within the last decade, the use of non-destructive testing is increased in the design phase. Unfortunately, the results are not used in the overall management process, because the formal decision of intervention is already made.

2.2.1 Degradation modelling

The degradation of a bridge condition describes the process by which one or a set of elements lose value with time (Sánchez-Silva & Klutke, 2016). In PONTIS, the future condition states of visual inspections were processed with the Markov chain method, depending on the assumption whether the interventions are performed during the time frame between inspections (Thompson *et al.* 1998), but this module has not been used in Estonia and degradation modelling has been based on more simple approaches like linear decay rate and the relation between average condition and annual decay rate.

The linear decay rate was defined during the implementation phase and revised after every bridge was inspected twice. The initial linear decay rate of CI was 0.8 and it was reduced afterwards with the last change in 2013 when 0.6 was suggested. The linear decay rate means, that if no intervention is done then bridge CI will annually decline the specific amount.

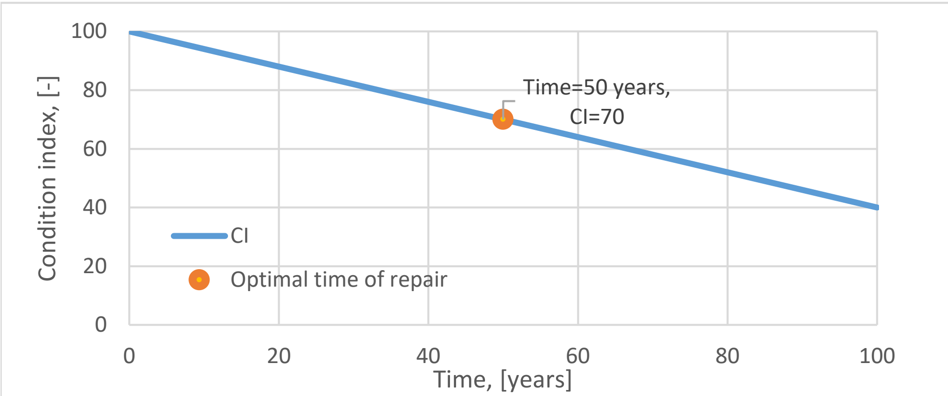

Figure 12. Condition change with linear decay rate 0.6 used in the BMS system.

The number was the same for all bridges despite the differences in typology, age or construction materials. The simplified approach was justified with the lack of data and expert knowledge.

In 2012, one Master student of Tallinn University of Technology, Andreas Papp (2012), analysed the BMS inspections and presented that a novel approach for condition degradation prediction is needed. The proposal was to relate the CI with the annual decay rate within specific groups based on the annual decay rate. The analysed results showed a good connection between these two indicators, meaning that bridges in better condition have a lower annual decay rate (Figure 13).

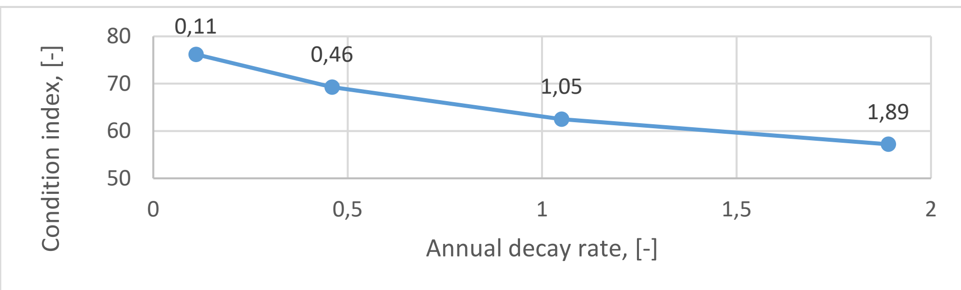

Figure 13. The relation between average CI and annual decay rate (Papp, Andreas, 2012).

Although the applicability of the approach was not presented in the thesis, it encouraged ETA to update the degradation modelling and change the linear decay rate with a more advanced linear approach. In past years, the framework has been updated annually and used in the decision-making process. The development of more advanced degradation modelling methods has been postponed due to new software development.

2.3 Description of National bridge network

The Bridge Network of Estonian national roads consist of 1023 bridges, which are located only on national roads. The bridges are mostly short (71%), with a length less than 25 m and the longest bridge has a length of 420.8 m. The main construction material is concrete (77%), followed by steel (22%) and other materials. Most of the bridges are constructed more than 40 years ago and approximately half of the bridge network is built between 1950 and 1980, which means that they are constructed during the Soviet Era and most of the bridges are designed based on typical element catalogues and are mounted. The number of constructed bridges has increased after the year 2000, with the development of new intersections and highways (Figure 14).

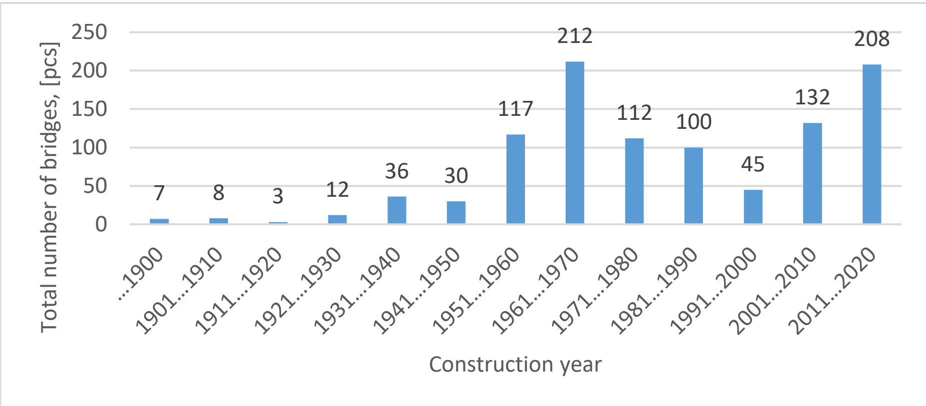

Figure 14. Overview of construction years of national bridges.

The average age of the national road bridge is 40.2 years and in 2020 the total amount of 27 was constructed or repaired.

To describe of Estonian bridge park based on an example of an average road bridge would be, crossing a river or other water obstacle, constructed in 1980, made from precast reinforced concrete elements with simply supported beams of a length of 14 m and will be repaired within next four years. The annual average daily traffic on a bridge is 2370 and the average time between interventions is 42 years.

All bridges can be divided into eight main categories according to the main girder, these are:

- Simply supported beam bridges (35% of all bridges).
- Simply supported slab (24% of all bridges).
- Rigid frame bridges (13% of all bridges).
- Culverts (flexible metal) (11% of all bridges).
- Arch bridges (8% of all bridges).
- Integral frame bridges (5% of all bridges).
- Cantilever beam bridges (2% of all bridges).
- Continuous beam bridges (1% of all bridges).
- Other (1% of all bridges).

Over the years, the most popular typologies have changed, but concrete has remained as the main construction material. In addition to physical parameters, bridges can be divided based on the load models used in the design process. More than 33% of bridges are already designed according to Eurocode load models (*EVS-EN 1991-2*, 2010), but a notable number of bridges are designed to resist loads used in the Soviet Union design codes. The speciality of the Soviet Union design codes was the extensive use of a typical combination of specific loads, which leads to catalogues of typical bridge elements. The most common bridges built between 1950 and 1990 are designed according to Typical Catalogues and some of the most known are listed below:

- Issue No 4 – catalogue for typical reinforced concrete or concrete slab superstructure (Mintransstroy, 1949).
- Issue No 31 – catalogue for typical small concrete bridges (Mintransstroy, 1955).
- Issue No 70 – catalogue for typical concrete pile supports (Mintransstroy, 1957).
- Issue No 56 – catalogue for typical prefabricated reinforced concrete beam superstructure (Mintransstroy, 1958).
- Issue No 123 and 123 addition – catalogues for typical prefabricated and tensioned concrete beams (Mintransstroy, 1959), (Mintransstroy, 1960).
- No 56 addition – catalogue for typical prefabricated reinforced concrete beam superstructure without diaphragms (Mintransstroy, 1962a).
- No 167 – catalogue for typical prefabricated reinforced concrete beam superstructure without diaphragms and higher steel grade (Mintransstroy, 1963).
- Series 3.505-12 – unified catalogues for precast and pre-stressed reinforced concrete superstructures (Mintransstroy, 1971).

Similar typical solutions are used in most of the former Soviet Countries including other Baltic countries Latvia and Lithuania.

In combination with a unified data collection method and with the help of statistical analysis techniques it is possible to extract information from a large bridge dataset for predictive purposes (Manyika et al., 2011) and make the overall management process more efficient. In Paper I principal component analysis (PCA) method was investigated. During the analysis, the elements were divided into 16 different groups (Table 4) and bridges were grouped within the specific typology. The division of elements was made based on available element observations and structural integrity, which describes the bridge structural performance where specific components have a bigger effect on the overall load-bearing capacity than others. The same division was used in other Papers related to the thesis.

Table 4. Element groups divided by position. From Paper I.

Non-structural elements	**No**	**Structural elements**	**No**
Overlay	1	Deck plate	9
Barriers	2	Edge beam	10
Handrails	3	Piles and columns	11
Drainage	4	Abutment cap, crossbeam	12
Slopes	5	Wing wall, front wall, abutments	13
Deformation joints	6	Diaphragms	14
Other (river bed, signs etc.)	7	Main girder	15
Waterproofing	8	Bearings	16

The average condition state of element groups of simply supported beam bridges shows clearly how the uncertainty in bridge management process affects the results.

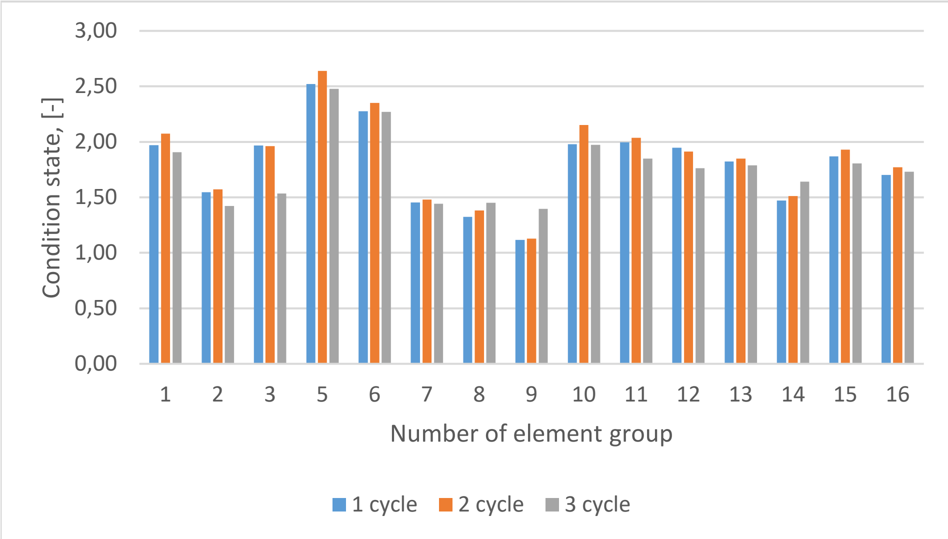

Figure 15. Average condition indexes of element groups for simply supported beam bridges. From Paper I.

Normally, due to the nature of the deterioration process, the element condition state should increase and decrease of the condition state (improvement of the condition) can be caused by the subjectivity of the inspector or unrecorded intervention. The differences between the condition state of the second and third cycle are much more significant, and they describe the situation where most of the elements are getting into a better condition state without any intervention. The differences are also emerged because of insufficient information, caused by the situation where not all bridges are inspected three times.

Although the primary purpose of the PCA is to redefine the input variables by reducing the amount of data while preserving the main information of the original dataset (Hotelling, 1933), (Jolliffe, 2002). A principal component can be defined (Eq. *12*):

$$Y_i = \alpha'_i x = \alpha_{i1}x_1 + \alpha_{i2}x_2 + \ldots + \alpha_{in}x_n = \sum_{j=1}^{p} \alpha_{ij}x_j, \tag{12}$$

where $\alpha'_i x$ – a linear function of the elements of principal component i, x is the maximum variance and α is a vector of p coefficients α. The first principal component shows the dataset with the largest variation and the second principal component is determined based on the orthogonal variance of the first principal component (Ringnér, 2008), (Abdi & Williams, 2010). For a better comparison of PCA results, it is possible to use the sum of the square of principal component coefficients α_{i}, because the sum of coefficients is equal to unity (Hanley et al., 2015). For comparison, coefficient based weighing factors were calculated (Eq. *13*):

$$\zeta = \sum_{j=1}^{p} \lambda_j x_j, \tag{13}$$

where ζ is a combination of weighting factors based on $\lambda_j = \alpha_{1,j}^2 \cdot 100\%$ and original condition ratings x_j.

The result of the analysis shows that despite the bridge typology first principal component was for the non-structural elements indicating most advanced damage for types in handrails, barriers or other elements, indicating elements with a shorter life cycle (Table 5). The variation can also be explained with superficial visual inspections because these elements are non-structural and are considered as irrelevant elements of a bridge because they incorporate minimal risk to structural load-carrying capability. Since the retained variation it is necessary to include results from them in intervention planning.

Although due to missing consideration of the risk of failure these findings should not be used directly in the bridge management process and deterioration models the statistical analysis show that it is incorrect to obtain weight factors based on only construction material or typology and additional expert judgement should be considered.

For future research, it is essential to cluster bridges based on a similar number of variables and add circumstances as additional weighting factors, that helps to provide relevant information without prioritizing common elements.

Table 5. Weighting factors λ of different bridge typologies. From Paper I.

Element group	Typology				
	Simply supported beam	Slabs	Slab in fragments	Cantilever	Rigid frame
1	0.64%	8.84%	1.92%	2.92%	1.79%
2	2.08%	62.27%	7.70%	9.09%	5.09%
3	77.93%	16.95%	72.85%	56.68%	NA*
4	NA*	NA*	NA*	NA*	NA*
5	4.16%	2.51%	1.36%	8.10%	2.27%
6	4.70%	NA*	NA*	1.25%	NA*
7	0.34%	4.51%	6.26%	3.99%	86.08%
8	0.10%	4.15%	9.25%	2.07%	0.33%
9	0.03%	0.00%	0.27%	0.13%	0.01%
10	0.17%	0.20%	0.09%	0.36%	0.65%
11	0.10%	NA*	0.20%	0.29%	0.38%
12	0.24%	NA*	0.03%	NA*	NA*
13	0.31%	0.57%	0.06%	3.20%	3.39%
14	0.05%	NA*	NA*	0.04%	NA*
15	1.93%	NA*	NA*	5.52%	NA*
16	7.21%	NA*	NA*	6.34%	NA*

Note: *data not available

3 Improvement of condition assessment

In addition to better classification of bridges and elements, it is important to increase the reliability of collected data. The addition of non-destructive testing to visual assessment has been suggested in many research projects including BRIME (Woodward et al., 2001), COST 345 (O'Brien et al., 2005), Sustainable Bridges (Bien et al., 2007), Mainline (Paulsson et al., 2014), Long Term Bridge Performance (Hooks & Weidner, 2016) and COST TU1406 (Wenzel & Pakrashi, 2019). In Papers II, III and IV the improvement of visual inspection-based condition assessment is investigated. The visual inspection will remain the most common assessment method because it is simple and efficient, but several non-destructive test methods could be integrated into the general bridge inspection procedure that would improve the quality of acquired data without eliminating the advantages. The integration of NTDs incorporates identification of the appropriate methods (Paper II), merging of the obtained results of NDT and condition assessment and verification of results in decision-making process (Paper IV).

3.1 Criteria for suitable NDT

To determine if the NDT is suitable to be included in the general inspection process a variety of different criteria can be used. The current selection of the criteria is based on the literature review and COST TU1406 Memorandum of Understanding (COST Action TU1406, 2015), where it is suggested that the criteria should be classified as measurable, but also descriptive criteria are used. For measurable criteria, a specific value can be determined, but descriptive criteria can be partially subjective since it is not possible to define it precisely. The description of the criteria selected in Paper II is given in Table 6.

Table 6. Criteria to be used for NDTs assessment. From Paper II.

Criteria	Description
Results' reliability (RR)	Descriptive criterion: It defines the reliability or accuracy of the results/measurements. It deals with the technological perfection of the investigation (accuracy) and the sensitivity of the method to various external factors.
Standardisation (S)	Measurable criterion: If there is a standard prescribed for the NDT under consideration, the results should be more reliable.
Usability (U)	Measurable criterion: It defines the number of parameters that can be measured with the NDT under consideration. Ability to investigate two or more materials, different types of damages or defects and similar.
Test duration (TD)	Measurable criterion: It defines the speed of the NDT execution and the speed of data acquisition. The criterion is predominantly related to the time spent by the inspector but can also be related to possible traffic disruption (bridge or individual traffic lane closure) due to the investigation.
Results' interpretation of complexity (RIC)	Descriptive criterion: it relates to the obtained raw measurements and the need for long and demanding analysis to obtain results (computer equipment and experienced engineers needed).
Cost (C)	Measurable criterion: It defines the cost of equipment acquisition, the cost of test execution and the cost of data analysis.

All of the criteria need to be considered, but to evaluate the NDTs the relative importance is also determined using the Analytic Hierarchy Process (AHP) introduced by Saaty (1990). The comparison of criteria was conducted on a nine-level descriptive scale, where experts compared the selected criteria in pairs and gave their judgements. The relative importance of each criterion was determined as the average value of all ratings attributed to the individual criterion by the experts.

The most important criterion is the reliability of the results, followed by duration indicators including inspectors time-on-site and office. For the test duration criterion, the classification of the selected NDTs is to a certain extent subjective, because in some tests the time of test is short, but if these tests provide local results, then it needs to be performed numerous times to provide comprehensive results, while others require more time to be performed, but determine the state of construction as a whole for the parameter measured (e.g. infrared thermography). A good example of NDT with high reliability and good duration is the phenolphthalein test as it provides reliable data regarding the carbonation depth at one location, but to know the condition of the overall structure, more than one test needs to be done. The cost criterion deals with equipment acquisition, maintenance, software cost and possible additional equipment needed for testing. The cost of acquisition is highly dependent on the technical characteristics of the equipment, therefore, the assessment based on this criterion is to some extent subjective. Usability includes the overall use of one method or equipment related to different damages or material properties. The criterion is less important because experienced inspectors are rarely surprised with the current condition of the structure and most tests are pre-determined in the office. The least important criterion was standardization because although it gives some confidence it does not guarantee reliable results. Scoring and description of criteria evaluation are presented in Table 7.

Table 7. Evaluation description of NDTs. From Paper II.

Criteria/ weight	**Scoring description**		
	3	2	1
RR/0.280	High, external conditions do not affect the results	Moderate, various factors can affect the results	Low, complementary investigations needed to confirm the results
TD/0.232	Short, total bridge inspection time is not noticeably increased	Moderate, total bridge inspection time is prolonged	Long, total bridge inspection time is doubled
RIC/0.170	Immediate results	Short analysis required	Prolonged analysis and high professional qualification necessary
C/0.134	Low	Moderate	High
U/0.108	Investigation of various materials and their parameters possible	Investigation of one material and two of its parameters possible	Limited usability, only one parameter is investigated
S/0.075	EN standard	National standard available	No relevant standard

The scoring is based on the expert judgement of authors and should reflect the importance of every criterion in the evaluation process of suitable NDTs.

3.2 Selection and evaluation of NDTs

The NDTs are selected based on previous literature review and expert judgement of COST Action TU1406 members. Methods are clustered in three groups to distinguish the objectives of test methods. The most common purpose of NDT is to define or specify a material property similar to the ones used in the design phase. Regarding the condition state of an element, several methods are dealing with the damage and defect assessment and separate field of NDTs are related to corrosion detection and assessment. Detailed evaluation of selected NDTs is presented in Table 8.

Table 8. NDT evaluation by the selected criteria. From Paper II.

NDT method	**Scoring ($V_{c,i}$)**					
	RR	TD	RIC	C	U	S
Cover measurement	3	3	3	3	1	2
Phenolphthalein test	3	2	3	3	1	3
Probe penetration test	2	3	3	3	1	2
Pull-off test	3	2	3	2	1	3
Rebound hammer	1	3	3	3	1	3
Impact echo	3	2	1	2	3	2
Thermography	2	2	1	2	3	1
Ground penetrating radar	2	2	1	1	3	2
Acoustic emission	2	2	1	2	2	2
Ultrasonic pulse echo	2	1	1	2	3	1
Half-cell potential	2	2	2	2	1	2
Galvanostatic pulse	2	2	2	2	1	1
Electrical resistivity	2	2	2	2	1	1
Linear polarization resistance	2	2	1	2	1	1

The final ranking of individual NDT is determined with the utility function (Eq. 14):

$$U_i = \sum_{c=1}^{m} V_{c,i} \cdot w_c, \qquad (14)$$

where *i* marks the considered NDT method, *c* is the defined criteria, $V_{c,i}$ is the scoring value of criteria *c* for NDT under consideration and w_c is the weight of criteria. Results are classified into three groups based on the classification and presented in Table 9.

The results show that proposed methods dealing with material properties have a high utility rating and are suitable to use in addition to visual inspections. Overall, all tests are quick, have low cost and are undemanding to perform. Damage and defects assessment methods are less suitable, because of the high complexity of the results' interpretation, followed by the on-site test duration. The only method that could be suitable is Impact Echo, which is used mostly for crack measurements and is quick and undemanding

(Helmerich et al., 2007). Corrosion related methods are measuring similar characteristics interestingly have similar utility function values, unfortunately, these methods are not suitable to be used during regular bridge inspection, because of the limited usability and it is suggested to use them only when reinforcement is already exposed.

Table 9. Final classification of NDT methods. From Paper II.

Application	U_i	**NDT**
Material properties	2.71	Cover measurement
	2.55	Phenolphthalein test
	2.43	Probe penetration test
	2.42	Pull-off test
	2.22	Rebound hammer
Damage and defects	2.22	Impact echo
	1.86	Thermography
	1.83	Acoustic emission
	1.80	Ground-penetrating radar
	1.63	Ultrasonic pulse-echo
Corrosion	1.89	Half-cell potential
	1.82	Galvanostatic pulse
	1.82	Electrical resistivity
	1.65	Linear polarization resistance

3.3 Conversion matrix of NDT

Based on COST Action TU1406 Working Group 1 Technical Report, most data are obtained by conducting visual inspection as an index form (Strauss et al., 2016), which means that the decision-making process relies highly on condition assessment and to enhance the use of NDTs it is important to show the clear numerical connection between the current system and novel methods. The outcome of an NDT is normally a numerical result and to translate the result to a condition state, additional expert knowledge is needed. In addition to suitable methods, in Paper IV a table of suggested threshold values for a few most used NDT in Estonia (Table 10) was presented. Since the table is based on expert judgement and available information in the Estonian context, then values can be conservative, because the work is not complete and variables may be expanded if additional NDT is available. Methods should be verified with benchmarking tests as the results are mostly dependent on other characteristic values.

The NDT methods in Table 10 are related to standardized procedures as follows:

- Sclerometer tests are carried out according to standard EN 12504-2 (EVS, 2012) and rebound values are converted to compressive strength according to curves suggested by equipment producers.
- Carbonation measurements are carried out according to standard EN 14630 (EVS, 2006).
- Chloride content measurements are carried out according to EN 14629 (EVS, 2007).
- Electrical resistance of a concrete is measured using method described in RILEM TC 154-EMC (Polder et al., 2000).

- Cover depth measurements are done using electromagnetic covermeter according to recommendations in BS 1881-204 (BS, 2020).

Table 10. Combination of NDTs and visual inspections at the element level. Partially from Paper IV.

Condition state,	Sclerometer [MPa]	Carbonation [mm]	Chloride content [ratio to limit state]	Resistance [kOhmcm]	Cover depth [ratio from limit state]
1 – very good	Median value more than normative compressive strength (f_{ck})	Average measured value less than 25% of the cover	Average measured value less than 100% of the threshold	Above 100	Average measured value is more than 100% of the threshold
2 – good	Median value 99% to 95% of f_{ck}	Average measured value between 26% to 50% of the cover	Average measured value between 101% to 150% of the threshold	Between 50 to 100	Average measured value is between 99% to 75% of the threshold
3 – poor	Median value 94% to 80% of f_{ck}	Average measured value between 51% to 100% of the cover	Average measured value between 151% to 200% of the threshold	Between 10 to 49	Average measured value is between 74% to 50% of the threshold
4 – very poor	Median value less than 80% of f_{ck}	Average measured value more than 100% of the cover	Average measured value more than 200% of the threshold	Less than 10	Average measured value is less than 50% of the threshold

To present the use of conversion matrix and draw attention to the difference of the potential outcome as intervention activity for one specific element group, several test protocols have been investigated and bridges have been tested in past years. The data presented in Table 11 were collected from the main girders of nine simply supported beams bridges introduced in Chapter 2.3. Bridges are designed according to Soviet Union design Catalogue Issue number 56 addition (1962a) or 167 (1963). The condition states of inspections (CS) are calculated based on the assessment result of element units and, for carbonation depth, the result is interpolated based on the threshold values of condition states. Possible intervention activities (INT) are regular maintenance (M), repair (Rep) or renovation/renewal (Ren).

Table 11. Comparison of condition states of different assessment methods. From Paper IV.

Bridge No	**Cons-truction year**	**Recons-truction year**	**Inspection-based**		**Carbonation depth**		**CS differr ences**
			Time of inspec-tion	CS – INT	Avg., [mm]	CS – INT	
883	1989	2002	2015	1.1 - M	6.9	1.4 - M	+0.3
826	1969	2010	2019	1.1 - M	18.7	2.3 - Rep	+1.2
907	1967	2001	2019	1.2 - M	20.0	2.8 - Rep	+1.6
911	1970	1998	2019	1.3 - M	28.0	2.8 - Rep	+1.5
908	1967	2001	2019	1.7 - M	13.6	2.6 - Rep	+0.9
503	1969	-	2019	2.1 - Rep	26.2	3.5 - Ren	+1.4
252	1965	2000	2018	2.3 - Rep	3.2	1.2 - M	-1.1
909	1974	-	2019	2.3 - Rep	37.5	3.5 - Ren	+1.2
306	1969	-	2019	3.0 - Ren	10.8	2.2 - Rep	-0.8

Only one bridge has the same potential intervention outcome, but generally, different assessment types have different outcomes and visual appearance-based condition state tends to give better outcomes and trigger intervention activity later. The obtained condition states should be used together with deterioration models, introduced in Chapter 4.3.

4 Uncertainty in performance assessment – benchmarking

Assessment related data has always some amount of uncertainty, which may impact management decisions and although there is no common knowledge on how to incorporate uncertainties in decision-making relevant decision requires the appropriate understanding and management of uncertainty. To assess the impact of uncertainty on the decisions, the uncertainty of assessments should be quantified. Uncertainty evaluation can be divided into two groups – a statistical measurement-based (Type A) and expert judgement based (Type B). In Paper III process and results of benchmarking tests were introduced and discussed. Also, Paper III and Paper IV investigated the influence of assessment uncertainty in degradation models.

4.1 Data collection

The benchmarking data was collected using the visual inspection methodology of ETA introduced in Chapter 2.2 and the most common NDTs used in Estonia: rebound hammer, carbonation depth and rebar depth measurement. Besides, two groups with different levels of expertise were involved, varying from students without any expertise to bridge inspectors with more than two years of experience.

The visual inspections were carried out on simply supported beam bridges designed according to Soviet Union design Catalogue Issue number 56 addition (1962a), 167 (1963) or 122 (Mintransstroy, 1962b) with different element condition states and condition index, which varied from 44.0 to 65.0. The test inspection involved two groups of inspectors, where the first group of seven inspectors consisted of bridge experts with different backgrounds in bridge engineering and the second group of five inspectors without any expertise. In the first group, only three inspectors had previous expertise in the bridge assessment and familiar with the methodology, other inspectors had only read the inspection manual, but done bridge assessments previously. The first group inspected two bridges, Lagedi and Assaku viaduct. Second group consisted of engineering Master students, who had no experience in bridge assessment, they inspected Saku viaduct.

NDT was carried out on not common simply supported slab bridges with good accessibility and also involved two groups of inspectors like in visual assessment. The first group consisted of bridge experts with previous experience and knowledge in testing and the second group consisted of eight engineering Master students, who had no previous experience in NDT, and they tested only the Alliku bridge.

4.2 Benchmarking results

The results of visual inspections are presented in Table 12 with every inspector rating to bridge condition, overall mean, standard deviation (SD) of assessments and previous CI. The overall mean of the Lagedi viaduct is close to the previous inspection result and the assessment of six inspectors refers to the condition where reconstruction is needed. Assaku bridge had a higher score for condition and overall inspection results were more scattered. Saku viaduct was inspected by inexperienced inspectors and although the mean value is close to the previous inspection result, the assessment of one inspector suggests closure of the bridge, one reconstruction and three assessments refer to the condition where only maintenance is needed.

Table 12. Visual inspection Condition Indexes. From Paper III.

Inspector	**Condition index**		
	Lagedi	Assaku	Saku
1	48.9	66.9	49.3
2	36.7	66.7	74.0
3	38.2	66.8	19.3
4	36.7	87.0	76.1
5	46.0	75.1	70.1
6	39.8	87.0	-
7	55.3	88.3	-
Mean	43.1	76.8	57.8
SD	6.6	9,6	21,5
Previous CI	44.0	65.0	57.0

The SD of all results shows that results are more scattered if the bridge is in better visual condition or inspectors are inexperienced. Nevertheless, if the inspector is not experienced then using only CI can lead to the wrong decision. Test results of NDT benchmarking are presented in Table 13, where Test results of experts are marked with E and student results are marked with S.

All results have relatively high accuracy compared to visual inspections and also all results showed that the bridge is in good condition. The carbonation depth test results are scattered because the investigated test method requires decent cleaning of the drill hole and measurement should be taken within 30 seconds after the solution is applied. Concrete cover results are in one range and all measurements were in the range between 38 to 50 mm.

Table 13. Test results of Alliku bridge. From Paper III.

Inspector	**Rebound value Q [-]**	**Carbonation depth [mm]**	**Average cover depth [mm]**
E 1	66.5	9	45
E 2	67.7	8	45
S 1	66.1	6	43
S 2	63.7	7	40
S 3	64.5	7	41
S 4	62.8	7	45
S 5	63.8	6	42
S 6	67.3	7	46
S 7	66.6	7	43
S 8	65.2	7	41
Mean	65.4	7.1	43.1
SD	1.6	0.83	2.0

In conclusion of NDT benchmarking tests, it is clear, that errors are smaller in comparison to visual inspections and non-experts can achieve higher accuracy without any previous experience in simple tests. It is still important to draw point out that the accuracy of the test method is not the only criterion, and a suitable test method should be selected considering the condition assessment outcomes.

For a better comparison of outcomes, the standard deviation of different assessments is expressed as a Coefficient of Variation (CoV) (Eq. 15):

$$CoV = \frac{\sigma}{\mu}, \tag{15}$$

where σ is the standard deviation and μ is the mean value of the measurand. Results are multiplied by 4 to present the difference of methods in relevance to condition states and presented in Table 14.

Table 14. Test results of benchmark testing. From Paper III.

Method	CoV*4
Visual inspection – students	1.49
Visual inspection – engineers	0.61
Visual inspection – experts	0.50
Rebound hammer	0.10
Carbonation	0.47
Cover depth	0.19

The highest uncertainty comes with visual inspections carried out by Master students, which is almost three times higher than experienced inspectors. The lowest uncertainty of assessment comes with the rebound hammer, but due to the low reliability of the method, it is not suggested to use this assessment separately from others.

4.3 Degradation modelling

Degradation modelling is used because the assessment of a condition individually does not allow long-term planning. Regarding uncertainties, then probabilistic modelling is preferred because it helps decision-maker to quantify and combine the impact of uncertainties of assessment and predictive models. The degradation models were calculated using continuous-time Markov processes based on assessment results of specific bridge element group. The degradation models were verified using a goodness-of-fit test under the assumption that the goodness-of-fit with 5% significance follows a χ_n^2 distribution under the assumption, where models are considered correct if the probability of goodness-of-fit is better than 5% of the sample value. The results with a discrepancy limit of 16.92 are presented in Table 15.

Based on the results presented in Table 16, only the non-structural element group designated "Other", did not pass the test, which means that there is adequacy between the sample and the model of all other element groups. An example of a verified transition intensity matrix is based on the element group "Main girder" (Eq. 16):

$$Q_{beams} = \begin{bmatrix} -0.0100 & 0.0100 & 0 & 0 \\ 0 & -0.0237 & 0.0237 & 0 \\ 0 & 0 & -0.0137 & 0.0137 \\ 0 & 0 & 0 & 0 \end{bmatrix}, \tag{16}$$

Based on the presented values, an average condition degradation profile, standard deviation of model and probabilistic condition with 95% confidence level is calculated (Figure 16).

Table 15. Goodness-of-fit test of deterioration models. From Paper IV.

Non-Structural Elements	Observations	Discrepancy (T)	Structural elements	Observations	Discrepancy (T)
Overlay	222	1.87	Deck plate	253	1.34
Barriers	198	4.37	Edge beam	189	1.21
Handrails	169	0.69	Piles and columns	64	0.78
Drainage	43	0.13	Supporting beam	212	4.73
Slopes	231	2.31	Wing wall, abutments	203	2.42
Deformation joints	177	0.67	Diaphragms	155	0.85
Other (riverbed, signs etc.)	155	29.79	Main girder	215	1.06
Waterproofing	253	16.75	Bearings	121	0.89

Due to the low transition probabilities, the average degradation profile is almost linear which makes the scheduling of intervention activities unrealistic, because with 100 years the condition changes from 1.00 to 2.24. Considering the errors of the model with 95% of confidence, the service life of concrete beams without intervention is 93 years.

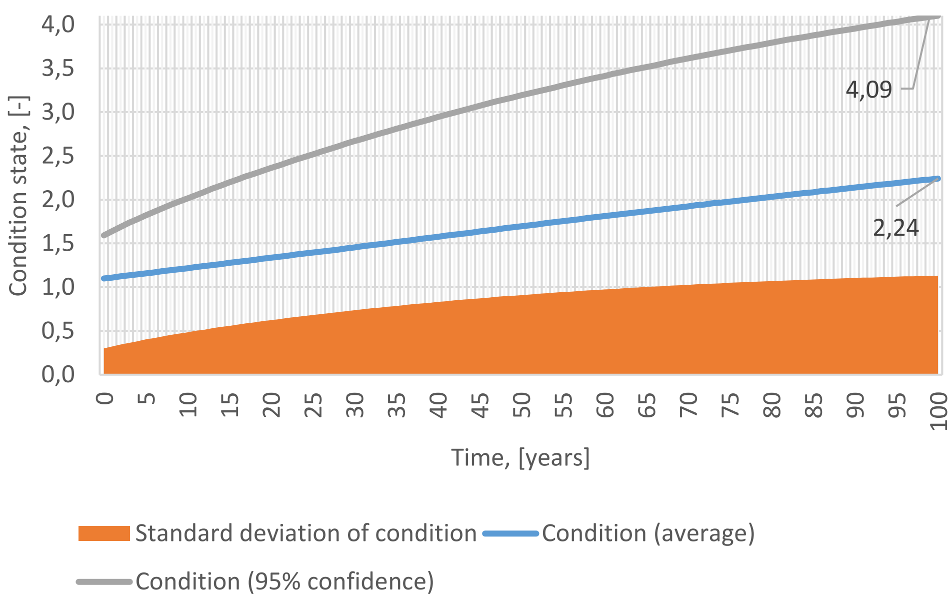

Figure 16. Condition degradation profile of concrete beams. From Paper IV.

The combination of initial deterioration model and condition assessment is done with Bayesian updating with informative prior. This approach has been introduced by Neves and Frangopol (2005), by combining Bayesian updating with simulation for improving expert judgment. The results of updating showed a significant impact on linear deterioration modelling. Based on the Bayes theorem, The probability density function of an updated condition is based on the Bayes theorem (Eq. 17) (A. H. Ang et al., 2015):

$$f^{"}(C_T) = K \times L(C_T) \times f^{'}(C_T), \tag{17}$$

where $f^{"}(C_T)$ is the probability density function of the condition at the time T considering both inputs, that are present in posterior distribution, $f^{'}(C_T)$ is the probability density function of the condition profile at the time T and $L(C_T)$ is the likelihood function considering the relativeness of the assessment. K is a normalizing constant for weight factor (Eq. 18) (A. H. Ang et al., 2015):

$$K = \frac{1}{\int_{-\infty}^{\infty} L(C_T) \times f^{'}(C_T) dC_T}, \tag{18}$$

The results were obtained with a Monte-Carlo simulation. The mean μ_C^{τ} and standard deviation σ_C^{τ} of a condition at time τ, can be calculated as Eq. 19 and Eq. 20 (M.-H. Chen et al., 2012) (L. C. Neves & Frangopol, 2008):

$$\mu_C^{\tau} = \frac{\sum_{i=1}^{n} C_{\tau}^{i} \times L(C_T^{i})}{\sum_{i=1}^{n} L(C_T^{i})}, \tag{19}$$

$$\sigma_C^{\tau} = \sqrt{\frac{\sum_{i=1}^{n} C_{\tau}^{i} \times L(C_T^{i})}{\sum_{i=1}^{n} L(C_T^{i})} - \left(\frac{\sum_{i=1}^{n} C_{\tau}^{i} \times L(C_T^{i})}{\sum_{i=1}^{n} L(C_T^{i})}\right)^2}, \tag{20}$$

where C_{τ}^{i} is the CI at time τ connected to sample i, C_T^{i} C_{τ}^{i} is the CI at time T connected to sample i and n is the number of samples.

To present the difference of initial and updated model uncertainties, then the probability mass distribution of the same degradation model in year 42 is presented (Figure 17). The initial standard deviation of the model has increased from 0.30 to 0.85 and in comparison, if taken into consideration the average value of the inspected condition is 1.60 and the standard deviation is 0.25.

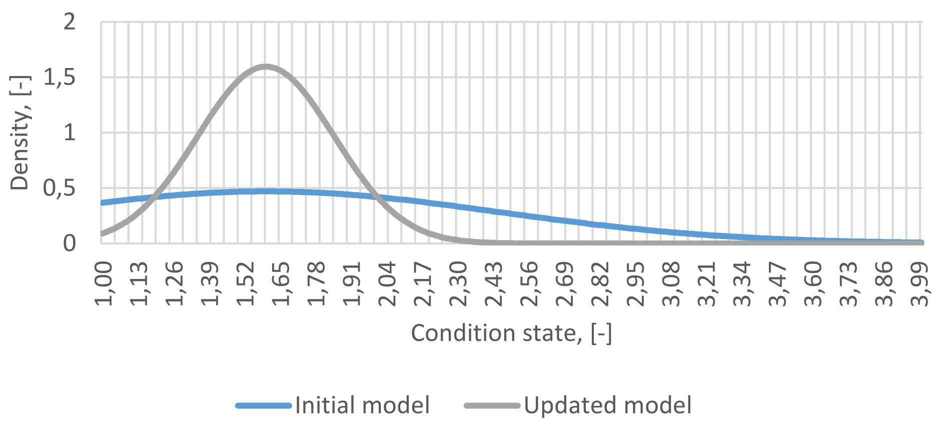

Figure 17. Probability mass function of initial and updated models on year 42. From Paper IV.

Based on Figure 17 it is clear, that model with updated information has a lower level of uncertainty. There is even a slight reduction of the uncertainty inspection is carried out by inexperienced inspector as presented in Paper III (Figure 18).

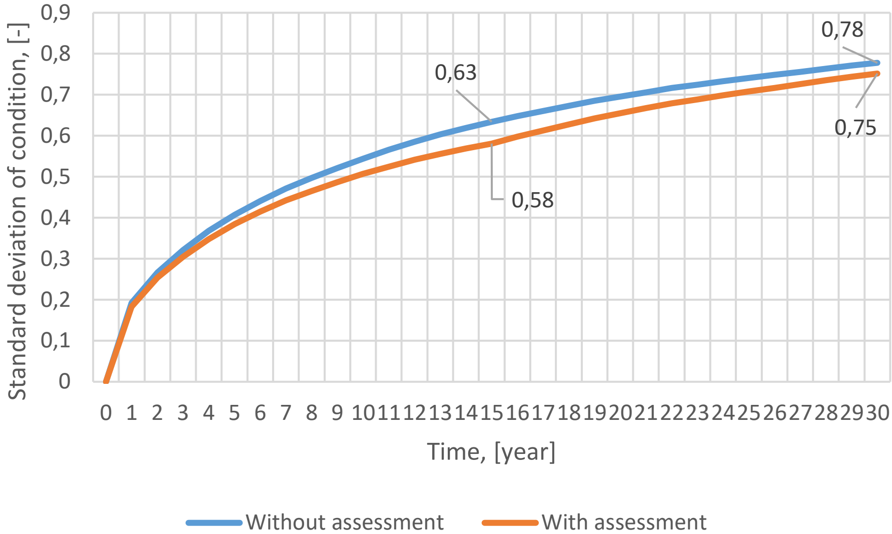

Figure 18. The standard deviation of condition with visual assessment carried out by an inspector without any previous experience. From Paper III.

If the result of the assessment is considered true, then more precise assessments can help to postpone more costly intervention activities. The optimal assessment is discussed in the next chapter (Chapter 5).

5 Optimal condition assessment scheduling framework

The best estimate of the intervention activity is currently based on the assessment result and commonly the assessment is scheduled after a specific or fixed interval. Since the result of an assessment is described as a mean value, then this combination may produce unnecessary inspections or lead to a wrong intervention decision. Since human nature can be described with Gaussian distribution, then it is possible to add uncertainty based on the standard deviation of the assessment method to overcome the problem of inspection scheduling and point out the lack of precise information if needed. The input for uncertainty can be collected during benchmarking tests as introduced in Publication III or using pre-defined values based on expert judgement. In Publication IV the scheduling of condition assessment was investigated and a novel framework was introduced.

The overall process of the introduced framework consists of nine steps as follows:

1. Collection of historical information, manual filtering of element groups of typical bridges without registered interventions and preparation of input data using a spreadsheet program.
2. Calculation of transition probability matrices for main element groups using continuous-time Markov model and Monte Carlo simulation using MATLAB® software developed by Denysiuk et al. (2017).
3. Verification of degradation models using goodness-of-fit test introduced in Chapter 4.3.
4. Collection of additional information with inspections and non-destructive testing carried out by experts.
5. Calculation of condition profiles with updated information using MATLAB® software based on the Bayesian inference method proposed by Neves and Frangopol (2005).
6. Comparison of model output based on the assessment method using MATLAB® software.
7. Setting limits to condition state and uncertainties based on the policies and expert judgement of ETA.
8. Determination of the maximal time frame to the next intervention action by combining the deterioration model and uncertainties in the model using MATLAB® software.
9. Determination of optimal inspection intervals to keep uncertainty and condition state under the desired level using MATLAB® software and spreadsheet.

To investigate which of the proposed inspection scheduling gives optimal output two parameters, trapezoidal area of the uncertainty of condition profile and total costs for the agency, are compared. The trapezoidal numerical integration is used to calculate the approximate area of uncertainties during the designed service life of 100 years and to simulate costs for the agency, total costs of element inspection and potential intervention are summed.

The optimisation algorithm is proposed with the main goal is to maximise the time between inspections and keeping the level of uncertainty under the desired threshold value. Based on the goal, the outcome is the time of inspection or potential intervention triggered by uncertainty or condition. Besides, more advanced NDT based inspection can be triggered. Finding the solution is based on linear interpolation of condition profile with 95% confidence expressed in Paper IV (Eq. 21):

$$y = y_0 + \frac{(x-x_0)(y_1-y_0)}{x_1-x_0}, \qquad (21)$$

where $x_{0,1}, y_{0,1}$ are coordinates of two known points and x, y are the coordinates of unknown point. The known points are the time and error of predicted or updated condition and the unknown point is partially defined with the limit value. The limit values for condition state and uncertainty in the model are related to owners' policies or needs.

5.1 Case study

Case study of the inspection scheduling concentrates on data of the same pre-cast beams of most common bridges as introduced in Chapter 3.3. Different inspection scheduling and assessment methods are compared during the service life of 100 years using condition limit for triggering intervention. Also, the total number of inspections, intervals, overall costs, area of uncertainty and years when uncertainty is above a threshold value is observed.

The limit values for condition and uncertainty in the Case Study are based on the current maintenance policy of ETA and expert judgement under the assumption that both variables follow a normal distribution and have a confidence level of 95%. The following restraints were used:

- Condition state is limited to 3.0 (poor) because the elements look visually bad and most likely a renewal will be triggered within the next few years.
- Uncertainty of a condition state is only limited with 1.0 because this gives the confidence that the assessed condition stays within the limits of one value.
- The initial condition and uncertainties are included in the initial probability distribution. Initial condition accuracy is ±10% and probability distribution is *P(0)=[0.90 0.10 0.00 0.00].*
- The standard deviation of degradation model σ_{model}=0.50.
- Regular visual inspection with σ =0.50 costs 150 EUR.
- Cover and carbonation depth measurement with σ =0.40 costs 400 EUR.
- Advanced NDT assessment including chloride content and resistivity measurements with σ =0.3 costs 800 EUR.
- Intervention activity is the renewal of a beam during an overall bridge reconstruction and the cost is estimated 10 000 EUR.

Currently, the condition assessment has a fixed period of 4-year intervals. In the comparison of the assessment method, then more accurate inspection costs more, but in exchange, it is possible to reduce the level of uncertainty and extend the time between inspections. For example, in Figure 19, a theoretical situation is observed, where 55 years after the construction it is not possible to reduce the uncertainty of condition with visual inspection to extend the time between inspections without exceeding the limit value. With more accurate advanced testing, it is possible to reduce the uncertainty and extent the time to 6 years.

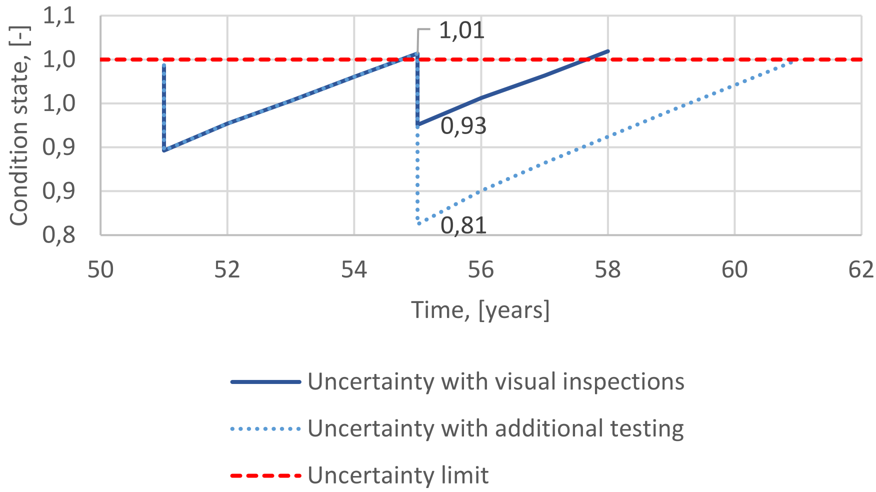

Figure 19. Uncertainties in year 55 with different assessments. From Paper IV.

Since the cost of visual inspection is low, then from the owner's point of view it is preferred if it is possible to continue with the current assessment regulation more optimal with better inspection scheduling. In Figure 20 current fixed 4-year interval inspection is compared with the optimised inspection interval.

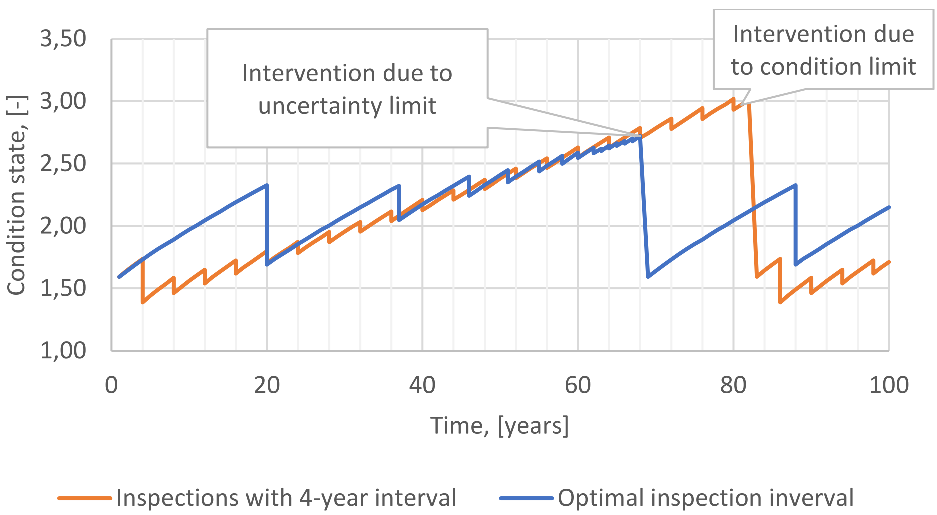

Figure 20. Condition profiles of 4-year and optimal interval visual inspections. From Paper IV.

Comparing the 4-year interval profile to optimised inspection interval then it is possible to see that the first 55 years the fixed interval triggers inspection too early. Although the optimal scheduling programme triggers 6 inspections with the annual interval in a row before potential intervention the difference in the total amount of inspections is 10. The limit of uncertainty in optimal inspection interval triggers intervention on year 69, which is earlier than the current system would trigger based on the condition profile, but without more accurate inspections the intervention is needed. If there would be an uncertainty trigger in the current system, then intervention would be triggered in year 59.

To increase the accuracy of performance assessment it is possible to add cover and carbonation depth measurement to all inspections and with this addition, the combined standard deviation of an assessment would be 0.40. In Figure 21 improved 4-year inspection interval is compared with the optimal inspection interval.

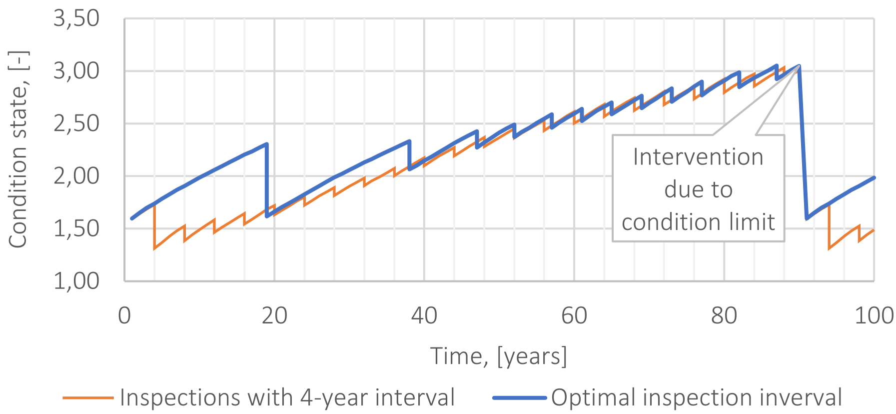

Figure 21. Condition profiles of 4-year and optimal interval inspections with additional testing. From Paper IV.

In both cases, the intervention is triggered in year 90 due to the condition limit and the main difference between the two approaches is the time between inspections and the number of total inspections. With a 4-year interval, there will be 22 inspections, but with the optimised approach, there are only 12 inspections with intervals ranging from 5 to 19 years.

In the optimal decision-making process, the overall costs are minimised and the situation where the assessment costs are higher than intervention should be avoided. The costs of assessment methods with current and optimised scheduling were compared during the service life of 100 years and an overview of the results is shown in Table 16.

With the current 4-year interval visual inspection-based approach, the inspections are triggered pessimistically in the first 55 years and afterwards, the level of uncertainty is above the threshold. Improving visual inspections with more accurate NDTs is also justified because after 55 years the time between inspections can only be extended with a more accurate assessment.

In opposite to the optimal framework, only the most accurate 4-year interval inspections can lead to a situation where no intervention, but due to high inspection costs the total cost is only 4.17% higher than the situation without any inspections. The lowest total cost is with visual inspections with the optimal interval. The area of uncertainty curve of the optimal approach is always higher than with the current system, but the uncertainty of the condition is always below threshold value 1.0, which means that inspections are triggered only when needed and not based on a strict schedule, which could lead to a wrong decision.

Table 16. Overview of total costs and uncertainties with different assessment approaches. From Paper IV.

Assessment type	No inspections	4-year σ =0.5	Optimal σ =0.5	4-year σ =0.4	Optimal σ =0.4	4-year σ =0.3	Optimal σ =0.3
Total number of inspections [-]	0	24	14	24	12	24	7
Total number of interventions [-]	2	1	1	1	1	0	1
Total cost [EUR]	20000	13600	12100	19600	14800	19200	15600
Area of uncertainty curve [-]	141.5	75.5	82.4	73.2	84.0	68.7	80.1
Uncertainty above 1.0 [years]	81	21	1	0	0	0	0

Based on the comparison the optimisation in overall can reduce the number of inspections but not the interventions and the overall costs compared to the *No inspection* scenario can save up to 39.5% (Optimal σ=0.5). In comparison of current and optimal approaches, the inspection costs can be reduced remarkably (minimally 1500 EUR). The other benefit compared to current inspection scheduling is the knowledge that uncertainty is almost always under the desired level and decisions can be made based on accurate data.

Conclusions

This thesis deals with different topics related to the condition assessment of existing concrete bridges in Estonia. Since the level of knowledge in bridge management under uncertainties is low in the local context, then a broader overview of the research field is presented in the beginning. The overall background of bridge management and decision making under uncertainties is based on the state-of-art literature review and details related to performance assessment, deterioration modelling and decision-making is described. The broader overview sets the scene for the current situation in the scientific field, where several different approaches regarding degradation modelling or decision-making have been used.

The aim was to introduce the uncertainties related to performance assessment, how time affects the indicator and propose a novel optimised approach for inspection scheduling. The focus of the optimisation was to keep the level of uncertainty under the desired level and maximise the time between assessments. The work can be divided into two main areas related to theoretical decision-making and practical performance assessment investigations.

The theoretical investigation involved statistical investigation of the Estonian bridge network, quantification of assessment results in deterioration model, updating with new potential information and combining threshold values with the optimisation algorithm.

Estonian bridge management principles are based on the PONTIS system, data is collected visually based on a strict time frame and many decisions are made using only condition information. The currently used performance assessment approach is simple, four different condition states are defined for elements, and the overall bridge condition index is calculated based on element ratings, units, and weight factors. There are 1023 bridges in the national bridge network, the average age of a bridge is 40.2 years, length 15 meters and more than half of concrete bridges are designed and constructed based on catalogues of typical elements. The statistical investigation of typical bridge element groups was done based on multivariate analysis using the basics of Principal Component Analysis, the analysis revealed that based on the change of condition state most principal elements are non-structural and the same elements groups have different importance if the bridge typology is changed. This means that it is incorrect to make decisions based on only construction material or typology and additional factors concerning the real influence of structural elements should be used.

The investigation of the deterioration model and inference process was done using stochastic continuous Markov chain models and Bayesian inference procedures with uncertainties. The models were verified using a goodness-of-fit test under the assumption that the goodness-of-fit with 5% significance follows a χ_n^2 distribution and only one element group results were not verified, which means that the used input data gives adequate results. The Bayesian updating process involved a combination of simulated degradation model data and expert judgement using Monte-Carlo simulation. The results showed that updating the model with uncertain data (CoV=1.49) gives more accurate information than using the only initial model. Overall, inspections help to reduce the uncertainty and potentially postpone the time to intervention.

The optimisation was based on the linear interpolation of the probabilistic condition profile with 95% of confidence. The main goal of the optimisation is to schedule inspections with the maximum possible time between events while keeping the level of uncertainty under the desired threshold value. As an additional result, the potential intervention can be triggered if there is a knowledge of the intervention measurand that minimises the costs of overall management processes. The latter optimisation was done only using simple data based on Estonian practice. The results show that using a 95% confidence condition profile in a situation where transition probabilities are low helps to keep the structure on the safe side by triggering interventions based on condition state limit instead of uncertainty limit. Compared to the 4-year inspection interval, optimal inspection scheduling keeps the level of uncertainty under the desired threshold value and trigger the inspections with a longer interval than the current approach without crossing the threshold value. Based on the main girder deterioration models, the currently used 4-year interval triggers inspections too quickly for 55 years and afterwards, the level of uncertainty is above the threshold, which means that the time of the inspection is not related to uncertainties involved in the assessment.

The more practical information presented help to form a framework that can be practically used in the condition assessment planning in Estonia. The improvement of the current visual inspection-based condition assessment is suggested to be done either with additional non-destructive testing or with more optimal inspection scheduling. To simplify the application procedure then a selection method of non-destructive testing methods was introduced and a conversion matrix for few common advanced assessment methods was proposed. For the selection of non-destructive testing methods, the highest utility ranking is for methods dealing with cover depth, material properties or impact echo method for crack measurement, but corrosion-related methods are not suggested. The difference of assessment outcomes using a conversion matrix is presented with carbonation information that was collected from the nine most common bridges. The results showed that visual inspection-based assessment tends to give better results and although the carbonation process has already depressed reinforcement, the visual appearance tend to trigger maintenance. Although improving visual inspections with more accurate non-destructive testing are suggested because the time between inspections can only be extended with a better uncertainty reduction of more accurate assessment, the need for additional testing is suitable after a certain period.

The theoretical background of quantitative uncertainty measurand based on the Guide to the expression of uncertainty in measurement (GUM), the mathematical method was supported with benchmarking tests of different assessment methods. The benchmarking was carried out on five reinforced concrete bridges and the overall procedure involved nine different experts and 13 novice students to find the upper limits of uncertainty in common assessment methods. The benchmarking tests revealed that although expert assessment is almost three times more accurate than students, the assessment of novice inspectors is better than the situation with no inspections. In the comparison of assessment methods, the most inaccurate non-destructive method, carbonation depth, is still better than visual inspection carried out by an expert.

Overall, the use of the proposed framework can be justified with the fact that the true condition of an element is always unknown and treating the uncertainty of the assessment as a probabilistic performance indicator triggering the inspection will ensure that the condition is known with needed uncertainty and unnecessary assessments are not triggered. This fact is now supported by the findings of the current thesis.

Suggestions for further research

While the focus of the current investigation was related to the uncertainties of performance assessment and decision-making related to inspection scheduling several areas require further research to make apply the uncertainty framework in intervention planning and management systems. The following topics are proposed for future research:

1. To provide more relevant information for intervention activities connected to structural performance of common bridge typologies and element groups it is essential to combine the condition models with risk-based models, which are commonly combined with reliability models.
2. To consider the actual physical phenomena of the concrete deterioration whilst keeping the current Estonian condition rating system the analytical models of carbonation, chloride ingress, frost-resistance and alkali-silica reaction should be analysed, and more relevant limit values for condition states should be proposed.
3. In the current work, the proposed optimisation framework is tested at the element level based on available methods and separate from the results of Principal Component Analysis. As a result, the framework is applicable to be used in one bridge inspection scheduling, but to use a similar approach in-network level, a more advanced element weight factor, available test method investigation is needed and the deterioration models of element groups should be combined.
4. Uncertainty related optimal inspection scheduling is compared with currently used regulation in Estonia, but to find the optimal solution considering the time of intervention, cost of intervention, cost of assessment, the importance of the structure and other threshold uncertainties, a multi-objective optimisation or similar method should be used and combined with Life Cycle Cost Analysis.

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Abstract

Development of an optimised condition assessment plan for common reinforced concrete bridges in Estonia

The dissertation concentrates on the optimisation of condition assessment, which is a common information collection method in bridge management. The uncertainty in the input data could lead to wrong management decisions, which will affect the overall performance of the bridge network. To overcome the limitation of qualitative visual assessments, a framework to optimise inspection scheduling for regular condition assessment of bridges is proposed. The framework is applied in the Estonian context but is usable in every management system because the whole process includes the definition of the measurand, quantification of accuracy in different assessments and comparing the novel approach with the current system. The main objective of the work is to keep the level of uncertainty under the desired level and maximise the time between assessments. The uncertainty is used because the real condition of an element is always unknown and using the uncertainty of the assessment as a performance indicator triggering the inspection will ensure that the condition is known with needed uncertainty and unnecessary assessments are not triggered.

The first part of the thesis is concentrated on describing the current situation in the field of decision-making and performance assessment. The differences in current practice and latest developments are notable because in the world, bridge management systems have been developed since the 1960s, but the Estonian system has been developed since 2004. Although Estonian development is based on the PONTIS system, it has been mainly used for the collection of condition assessment data. Also, the subjectivity of visual inspection is known in Estonia, but the existence of the uncertainties has not been recognised or investigated. In addition to the description of the current situation, a statistical investigation based on Principal Component Analysis was done and it revealed that based on the change of condition state most principal elements are non-structural, which means that making intervention decisions based on only construction material or typology is incorrect and additional factors concerning the real influence of structural elements should be used.

The information presented in the second part of this thesis form a framework that can be practically used in the condition assessment planning in Estonia. The improvement of the current visual inspection-based condition assessment is suggested to be done either with additional non-destructive testing, with more optimal inspection scheduling or the combination of both previous. To simplify the application procedure a selection of non-destructive testing methods was introduced and a conversion matrix for few common advanced assessment methods was proposed. For non-destructive testing methods, the selection of suitable non-destructive testing methods was also investigated, using Analytical Hierarchy Process. The highest utility ranking is for methods dealing with cover depth, material properties or impact echo method for crack measurement, but corrosion-related methods are not suggested. In addition to the proposal of a conversion matrix, the difference of assessment outcomes was investigated with carbonation information collected from the 9 most common concrete bridges. The results showed that visual inspection-based assessment tends to give better condition and although the carbonation process has already depassed reinforcement, the visual appearance tent to trigger maintenance instead of repair.

The quantification of uncertainty is based on the Guide to the expression of uncertainty in measurement (GUM) and is supported with benchmarking tests of different assessment methods. The results of condition assessment are modelled with continuous-time Markov models, verified with verified using χ_n^2 goodness-of-fit test, and updated with Bayesian inference. The benchmarking was carried out on 5 bridges and the overall procedure involved 9 different experts and 13 novice students to find the upper limits of assessment methods. The benchmarking tests revealed that although expert assessment is almost 3 times more accurate than students, the assessment of novice inspector is better than the situation with no inspections. In a comparison of assessment methods, the most inaccurate non-destructive method, carbonation depth, is still better than visual inspection carried out by an expert.

The optimisation framework for optimal inspection scheduling based on linear interpolation and the obtained results are compared with the current Estonian assessment regulation to show the efficiency of a novel approach. Based on the main girder deterioration models, the currently used 4-year interval triggers inspections too quickly for 55 years and afterwards, the level of uncertainty is above the threshold, which means that the time of the inspection is not related to uncertainties involved in the assessment. Although improving visual inspections with more accurate non-destructive testing is needed after a certain period in use, the time between inspections can only be extended with a more accurate assessment.

Lühikokkuvõte
Optimeeritud seisukorra kontrolliplaani välja töötamine tüüpilistele Eesti raudbetoonsildadele

Väitekiri keskendub sildade seisundi kontrollimisele, hinnangutega kaasnevale määramatusele ning ülevaatuste vahelise aja optimeerimisele. Seisukord on sildade haldamises üks tähtis muutuja, mida kasutatakse investeerimisotsuste tegemisel ja see tähendab, et algandmete määramatus võib viia valede otsusteni, mis omakorda mõjutavad sillavõrgu üldist toimivust. Maailmas on varasemalt määramatust käsitletud õigete parendustegevuste vaates ning uuringutes on keskendutud üldise haldusega seotud kulude optimeerimisele. Antud töös välja pakutud raamistiku loomisel on teadlikult piirdutud seisukorra ülevaatustega, et tähtsustada selle etapi käigus kogutud andmete olulisust sildade halduse kontekstis. Tulemuste paremaks iseloomustamiseks on uuringus arendatud raamistiku rakendamist täiendavalt piiritletud Eesti tüüpiliste raudbetoonsildadega, mille seisukorra andmeid on enam kui 10 aastat ilma põhjalike uurimusteta kogutud. Kuna kogu protsess on universaalne, sisaldades mõõdetava suuruse määratlemist ja mõõtetäpsuse numbrilist väljendamist erinevates hinnangutes, siis tegelikkuses saab seda kasutada ka teistes varahaldussüsteemides. Töö peamine eesmärk on juhtida tähelepanu seisundi hinnangutega kaasnevale määramatusele haldamises ja pakkuda välja lahendus, kuidas hoida määramatuse tase soovitud tasemest madalamal samal ajal pikendades hindamiste vahelist aega. Määramatust kasutatakse näidikuna seetõttu, et elemendi tegelik seisund ei ole kunagi täpselt teada ja kui hindamise protsessi täiendatakse määramatusega, siis tagatakse, et seisund on alati piisavalt täpselt teatud ja puuduvad üleliigsed kulud. Töö võib jaotada kahte ossa, millest esimeses on tutvustatud hetkeolukorda maailmas ning Eestis ja teises on keskendutud raamistiku loomisele.

Praeguse olukorra kirjeldamisel on peamiselt tähelepanu juhitud toimivuse hindamisele ja otsuste tegemisele maailmas ning Eestis. Erinevused on märkimisväärsed ja on seotud juba puhtalt asjaoluga, et maailmas on sildade haldamise süsteeme arendatud juba alates 1960. aastatest, kuid Eesti süsteemi hakati välja töötama alles 2004. aastal, millest on praeguseks korralikult rakendatud seisukorra hindamise ja andmete kogumise metoodikad. Visuaalse ülevaatusega kaasnevat määramatust on Eestis vähesel määral uuritud ning valdkonna spetsialistid on sellest teadlikud, kuid parendustegevustega seotud otsused põhinevad jätkuvalt subjektiivsel hinnangul. Selleks, et sillapargi hetke seisukorda kirjeldada, uuriti Eesti sildasid statistiliselt peamiste komponentide analüüsi erinevate algoritmide abil. Tulemused näitasid, et peamised komponendid ei ole kestvusega seotud ja eri silla tüüpidel on kõige olulisemad elemendigrupid erinevad, mis tähendab, et otsuste tegemisel ei ole korrektne lähtuda ainult silla tüübist või ehitusmaterjalist ning kasutusele tuleks võtta täiendavad tegurid, mis arvestaksid elemendi tähtsusega konstruktsiooni kestvusele.

Töö teises pooles on keskendutud tervikliku raamistiku eri osade tutvustamisele ja on toodud näited, mis näitavad, et seda on võimalik praktiliselt kasutada seisundi kontrollide planeerimisel. Hetkel Eestis kasutusel olevat seisukorra hindamist on soovitatud täiustada täiendava mittepurustava katsetamise, optimaalse seisukorra kontrollimise või kahe eelneva kombinatsioonina. Täiendavate mittepurustavate katsete kasutuselevõtu lihtsustamiseks tutvustati Analüütilise Hierarhilise Protsessil põhinevat sobivate katsemetoodikate valikuprotseduuri ja valikut erinevatest katsetest. Kõige enam sobivad süsteemi täiendamiseks metoodikad, mille abil saab hinnata materjali

omadusi, kaitsekihi paksust ja pragude sügavust, kuid korrosiooni hindamisega seotud metoodikad ei ole sobilikud. Lisaks metoodikatele töötati välja ülemineku maatriks, mille abil saab lihtsalt mõõtetulemuse ümber tõlgendada elemendi seisunditasemeks. Üleminekumaatriksi sobivuse näitlikustamiseks teostati mõõtmisi ja koguti andmeid 9 enamlevinud betoonist sillalt. Karboniseerumise ja visuaalse ülevaatuse tulemuste võrdlusest võib järeldada, et visuaalne hindamine annab parema seisukorra ehk vaatamata asjaolule, et karboniseerumine on jõudnud juba armatuurini, lubab visuaalne hinnang remondi asemel jätkata tavahooldusega.

Töö keskmes oleva määramatuse numbriline väljendamine põhineb metroloogiast tuntud mõõtemääramatuse üldteoorial ja erinevatel võrdluskatsetel. Rajatiste seisukorda ja sellega kaasnevat mõõtemääramatust modelleeritakse pideva ajaga Markovi mudelitega, mille seosed tegelikkusega on valideeritud Hii-ruut testi abiga. Uute andmetega arvestamiseks on prognoosimudeleid kombineeritud Bayes'i seadusega ja tulemused on saadud Monte-Carlo simulatsiooni abil. Erinevate metoodikate võrdluskatsed viidi läbi kokku viiel sillal ja erinevate mõõtemääramatuste leidmiseks kaasati kokku 9 eksperti ja 13 üliõpilast. Võrdluskatsete tulemusena selgus, et eksperdi visuaalne hinnang on kogenematu üliõpilase hinnangust ligi 3 korda täpsem ja ka kogenematu üliõpilase hinnang on parem, kui hinnangu puudumine. Eri katsemetoodikate võrdluses on kõige ebatäpsem karboniseerumissügavuse määramine, mis on vaatamata kõrgele mõõtemääramatusele ikkagi täpsem, kui eksperdi visuaalne hinnang.

Optimaalne seisukorra ülevaatuse intervall on leitud seisunditaseme ja määramatuse lineaarse interpoleerimise teel. Uudse lähenemise tõhususe tõestamiseks võrreldi tulemusi praegu Eestis kehtivate põhimõtetega, kus ülevaatused on ajastatud 4-aastase intervalliga. Võrdluse aluseks võeti tüüpiliste betoonsildade peatalade 95%-lise usaldusnivooga tagatud prognoosimudelid ning piirväärtused määratleti vastavalt omaniku soovidele. Võrdluse tulemusena selgus, et praeguse lähenemise puhul on 4-aastane intervall liiga sage ja ei taga piisavat täpsust. Esimese 55 aasta puhul võib ülevaatuseid ajastada vastavalt optimeeritud seisukorra kontrolli plaanile ning seejärel on vaja visuaalseid ülevaatuseid täiendada täpsemate mittepurustavate katsetustega.

Lõppkokkuvõttes leiti, et praegust seisukorra kontrolli on võimalik täiendada selliselt, et seisukorra hinnang jääb alati allapoole soovitud piirväärtust ning kombineerides töö käigus välja töötatud raamistiku erinevate parendustegevuste optimeerimisega on võimalik teha paremaid ja täpsemaid otsuseid.

Appendix I

Paper I
Sein, S., Matos, J. C., & Idnurm, J. (2017). "*Statistical analysis of reinforced concrete bridges in Estonia"*. The Baltic Journal of Road and Bridge Engineering, 12(4), 225–233.

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STATISTICAL ANALYSIS OF REINFORCED CONCRETE BRDIGES IN ESTONIA

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Abstract. This paper introduces a possible way to use a multivariate methodology, called principal component analysis, to reduce the dimensionality of condition state database of bridge elements, collected during visual inspections. Attention is paid to the condition assessment of bridges in Estonian national roads and collected data, which plays an important role in the selection of correct statistical technique and obtaining reliable results. Additionally, detailed overview of typical road bridges and examples of collected information is provided. Statistical analysis is carried out by most natural reinforced concrete bridges in Estonia and comparison is made among different typologies. The introduced multivariate technique algorithms are presented and collated in two different formulations, with contrast on unevenness in variables and taking into account the missing data. Principal components and weighing factors, which are calculated for bridges with different typology, also have differences in results and element groups where variation is retained.

Keywords: bridge management, condition assessment, multivariate analysis, principal component analysis, statistical analysis, visual inspections.

1. Introduction

Bridges play an essential role in an infrastructure network, a famous Serbian writer has introduced them as follows*: "Of everything that man erects and builds in his urge for living nothing is in my eyes better and more valuable than bridges. They are more important than houses, more sacred than shrines. Belonging to everyone and being equal to everyone, useful, always built with a sense, on the spot where most human needs are crossing, they are more durable than other buildings, and they do not serve for anything secret or bad."* – Andrić (2015), Nobel Prize Winner in 1961.

Due to the economic and societal risk of failure of bridges, it is vital for asset managers and stakeholders to implement adequate management systems to ensure the risk of failure is under stated performance criteria (Mueller, Stewart 2011). Many authorities, including Estonian Road Administration, have implemented a management system to monitor the condition of their existing bridge network (Lauridsen *et al.* 1998). A Bridge Management System (BMS) is a systematic and rational approach to perform all management activities related to managing a bridge stock (Scherer, Glagola 1994). It usually includes planning intervention activities to fulfil the serviceability requirements and using computational tool to track, record and process the results of management actions (Lauridsen *et al.* 1998). Bridge Management System comprises coordinated activities to realize their optimal value, which involves balancing of costs, risks, opportunities and performance requirements. This framework allows asset managers to plan further assessment and intervention (Matos *et al.* 2015) often rely on collecting condition ratings of structures using visual inspections (Das 1998; Estes, Frangopol 2005).

Unified visual inspections, as the most basic level in the assessment of existing road structures (Rücker *et al.* 2006), have been used on Estonian national road bridges for more than 10 years and more than 40 000 element condition states have been recorded during the period. Although there are only 995 bridges, it is difficult to discern, which bridge is the worst state or what kind of intervention it is possible to carry out. One solution, to make decision-making more efficient, it is possible to exploit modern computational methods to perform "big data" analysis, which allows the extraction of information from a large dataset for descriptive and predictive purposes, using statistical techniques (Manyika *et al.* 2011). The purpose of data-reduction in multivariate analysis is to represent the original data with suitable lower-dimensional space, which helps to enable visualisation and discover data structures and patterns (Martinez *et al.* 2010). This paper investigates the use and applicability of linear multivariate analysis method called Principal Component

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Analysis (PCA), where different algorithms are applied on collected visual inspection database of most common reinforced concrete bridge typologies in Estonia to see the possibilities of statistical analysis and to point out elements with higher variance.

Hanley *et al.* (2015; 2016) have previously investigated Principal Component Analysis applicability to condition rating data of BMS by integrating this technique into a network of road bridges in Ireland and Portugal. In conclusion, the PCA is indicated to be a viable tool in the assessment of large data sets relating to engineering applications, and it is possible to convert results directly to weighing factors of condition ratings. It was also suggested to investigate other typologies with more detailed description (Hanley *et al.* 2016).

In this paper, visual inspection database and most ordinary reinforced concrete bridges of Estonian national road network are introduced. The difference with previous research is an additional comparison of two algorithms of PCA and element groups with greater variance are pointed out. Two different algorithms are compared to show the possibility of making wrong decisions based on results of PCA. This paper attempts to address this issue by choosing appropriate indicators and giving a rational basis for modelling technique and decision-making. From an engineering perspective, decisions are naturally the result of a well-structured reasoning which justifies the selection of the final solution. Decisions made because of a logical, scientifically structured process are considered as rational decisions (Sánchez-Silva, Klutke 2016).

Since there is more than one possible decision, for the better outcome the use of statistical techniques should also be considered. Many techniques have been developed for this purpose, but PCA is one of the oldest and most widely used (Jolliffe, Cadima 2016). It is first formalized by Pearson (Pearson 1901) and described in its algebraic form by Hotelling (Hotelling 1933). The technique identifies the component ratings which are more important than the others regarding explained variability and reassesses these relative importance ratings of different factors from an engineering point of view, based on additional data (Hanley *et al.* 2015). Principal Component Analysis is dimensionality-reduction method where a set of original variables are replaced by an optimal set of derived variables, called Principal Components (PCs) (Jolliffe, Cadima 2016).

Table 1. Description of condition state ratings in Estonian BMS

Condition state rating	Description, action
1	Element in good condition, no rehabilitation needed
2	Element in fair condition, minor rehabilitation, and local repair needed
3	Element in bad condition, repair needed
4	Element in critical condition, major repair or replacement needed

One solution is to use the results as an input for the bridge condition calculation or predictive models for different typologies of bridges by calculating the importance of relevant elements which define the performance of individual bridge based on visual inspections. The results are also useful for comparing relative components of bridges with different typology, age, traffic intensity, exposures or other environmental situations.

2. Description of dataset

The average age of Estonian national road bridges is 40 years, which indicates the necessity to have an overview of their condition and to make correct decisions to preserve them as long as possible. The implementation of unified visual inspections and management in Estonian national roads started in 2003; the system was based on United States system PONTIS, which was initially used to monitor the state of existing bridge stock and to predict the future condition of bridges (Thompson *et al.* 1998). First official inspections were carried out in 2005, and within this period every bridge is visually inspected twice and 400 of them already three times.

Assessment of a bridge consists of inspecting every element unit of the bridge and evaluating each with a condition rating based on a scale of damage present and necessary rehabilitation method (Table 1). An overall element condition index is calculated based on the overall quantity of units and state factors, as shown in Eq (1):

$$H_e = \frac{\sum_s k_s q_s}{\sum_s q_s} \cdot 100\% , \tag{1}$$

where H_e – condition state of element; s is condition state; k_s – coefficient of state and q_s is the amount of units in current state. Bridge condition index is calculated based on element condition index and weight factor. The overall condition rating of the bridge, which is often the primary decision criterion of investments, has misleading impact because different states of element deterioration possess equal condition ratings in overall.

A central element in life-cycle assessment (LCA) involves making predictions about the degradation of the system. It requires a clear understanding of the physical laws which define the system behaviour and possible uncertainties. The degradation of a bridge condition describes the process by which one or a set of elements lose value with time (Sánchez-Silva, Klutke 2016). In PONTIS, the future condition states of visual inspections are processed with Markov chain method, depending on the assumption whether the interventions are performed during the time frame between inspections (Thompson *et al.* 1998).

From an overall number of 995 Estonian national road bridges, 778 are constructed of reinforced concrete. Although the number of reinforced concrete bridges has decreased in last decade, by replacing them with soil-steel composite bridges, it is still most popular construction material in Estonia.

There are 4 main typologies for reinforced concrete bridges, which are divided into 7 different groups according to main girder and construction year, these are:

1. Mounted simply supported beams with diaphragms (constructed after 1956).
2. Mounted simply supported beams (constructed after 1963).
3. Cast in site simply supported slabs (constructed before 1960).
4. Mounted or cast in fragments simply supported slabs (constructed after 1960).
5. Cast in site simply supported beams (constructed before 1956).
6. Mounted frames (constructed after 1978).
7. Cast on site simply supported cantilevers (constructed before 1956).

These typologies make 98% of all reinforced concrete bridges. The primary concentration is on bridges that were constructed or repaired before 2005, when first unified visual inspections were carried out, because every kind of intervention is changeing the natural degradation process. In last 10 years, the average amount of bridges being built or reconstructed is 41, and since the main criteria of multivariate analysis is to exclude bridges with intervention, there has been a filtering process before analysing most common typologies. After excluding repaired and reconstructed structures, only 5 main typologies with a dataset of 501 reinforced concrete bridges remain (Table 2). To be clear, Group No. 1 consists of both simple span bridge typologies constructed after 1956 and Group No. 4 consists of the simple span and cantilever bridges.

As mentioned before, the visual inspection results are recorded on an element basis. It is possible to select from 124 different elements, which are distinguished by type and overall dimensions. For example, there are 5 different elements for piers with different width.

Within this analysis, the elements are divided into 16 different groups (Table 3) to investigate overall patterns in specific typology. This division of element groups was made based on available element observations and structural integrity, which describes the bridge performance where structural components have bigger effect on the overall load bearing capacity than non-structural. The condition ratings for each group are calculated from every represented element condition and are on the scale of 1–4 in increasing order as condition rating for elements (Table 1). No weight factors are used in the calculation of group and condition ratings are average results of elements.

Another limitation of data usage is due to the peculiarity of bridge inspections, which are carried out in different time frame. Bridges have been inspected seasonally or in so-called cycles, where assessment of all structures is done in 3 or 4 years. The first cycle was in 2005–2007, second in 2010–2013 and the third one started in 2015. An example of average condition indexes of all inspection cycles is shown in Fig. 1, where calculated average conditions are shown.

Due to the nature of deterioration process, without any intervention, the element state has to be higher, and within the period between first two assessments, the condition index is higher or slightly better. Improvement of the condition is normally caused by the subjectivity of inspector or unrecorded maintenance works. Nevertheless, differences between condition state of the second and third cycle are much more significant, and they describe the situation where most of the elements are getting into better condition state without any intervention. The differences are mainly emerging because of insufficient information, caused by the situation where all the bridges are inspected twice, and only half of them three times. In further analysis, the primary attention is paid to 501 bridges with two different results.

3. Formulation of principal component analysis and algorithms in case of missing data

The primary purpose of the PCA is to reduce the dimensionality of a set of data and redefine the input variables as principal components (PCs). It is a linear combination of the original variables, having fewer variables than the original dataset while preserving most of the information (Hotelling 1933; Jolliffe 2002). The first principal component Y_1 is defined as Eq (2):

Table 2. Main typologies of Estonian national road bridges without interventions

Group No.	Type	Construction year	Amount
1	Simple span beam	1956–...	195
2	Slabs	...–1959	86
3	Slab in fragments	1960–...	118
4	Simple span and cantilever beam	...–1955	34
5	Frames	1978–...	68
	Total		501

Table 3. Element groups divided by position

Non-structural elements	Number	Structural elements	Number
Overlay	1	Deck plate	9
Barriers	2	Edge beam	10
Handrails	3	Piles and columns	11
Drainage	4	Abutment cap, crossbeam	12
Slopes	5	Wing wall, front wall, foundation, abutments	13
Deformation joints	6	Diaphragms	14
Other (river bed, snow nets, signs)	7	Main girder	15
Waterproofing	8	Bearings	16

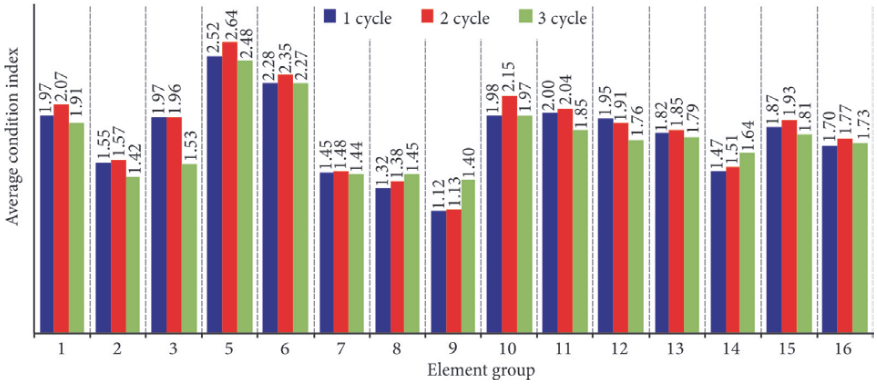

Fig. 1. Average condition indexes of element groups for simply supported beams (Group No. 1)

$$Y_1 = \alpha_1' x = \alpha_{11} x_1 + \alpha_{12} x_2 + \ldots + \alpha_{1n} x_n = \sum_{j=1}^{p} \alpha_{1j} x_j , \quad (2)$$

where $\alpha_1' x$ – a linear function of the elements; x having maximum variance; α – a vector of p coefficients α.

The first principal component is the direction along which the data set shows the largest variation (Ringnér 2008), and the second component is determined under the constraint of being orthogonal to the first component and to have the largest variance (Abdi, Williams 2010). The second principal component $Y_2 = \alpha_2' x$ is found in a similar manner to the first principal component, and so on for the subsequent principal components up to p PCs. Recommendation is given for the number for PCs accounting the variance in the data set, which must be significantly lower than all the calculated components (Jolliffe 2002).

As described by Hanley *et al.* (2015), it is possible to use the sum of the square of PC coefficients α_i for each variable, because the sum of coefficients are equal to unity and it is a better indicator in comparison of results. Weighing factors are derived from coefficients as shown in Eq (3):

$$\varsigma = \sum_{j=1}^{p} \lambda_j x_j , \quad (3)$$

where ζ – a combination of weighing factors based on $\lambda_j = \alpha_{1,j}^2 \cdot 100\%$ and original condition ratings x_j.

The previous formula, discussed in matrix terms, where a PCA is conducted through an Eigenvalue Decomposition (EVD) or a more robust and generalized Singular Value Decomposition (SVD) (Chambers 1977).

For a data matrix X of n observations on p variables measured by their means Eq (4):

$$X = ULA' , \quad (4)$$

where L – an $(r \times r)$ diagonal matrix; U and A – $(n \times r)$ and $(p \times r)$ matrices, respectively, with orthonormal columns, and r – the dimensionality of X.

SVD approach to PCA is shown to be computationally efficient and generalized method to determining the PCs.

As in previous investigations by Hanley *et al.* (2016) in using PCA for analysing BMS data, there is a problem with "missing data" in data-subsets, which describes a situation where a statistical difference of observations in among element groups appears. In multivariate analysis, it is often possible to use the existing structure of the data to estimate the missing data and complete a dataset, for example using Alternating Least Square (ALS) algorithm.

The algorithm alternates between imputing the missing values in and applying standard PCA to the in-filled (complete) data matrix (Jolliffe 2002). Initially, the missing values are replaced by the row-wise means of the previous matrix. The covariance matrix of the complete data are then estimated without the problems of principal components being more abdundant than estimated variances and situation where the covariance matrix is negative, as in Eq (5)–(6) (Ilin, Raiko 2010).

The Alternating Least Square algorithm alternates among the updates:

$$X = \left(A'A\right)^{-1} A'U' , \quad (5)$$

$$A = U'X'\left(XX'\right)^{-1} , \quad (6)$$

where X – $(p \times n)$ matrix of principal components; U and A – $(n \times r)$ and $(p \times r)$ matrices, as stated in Eq (3).

This iteration is efficient when only a few principal components are needed, so the number of principal components must be significantly lower than dimensionality of data vectors. (Roweis 1998). Alternating Least Square alternates among imputing the missing values in updated matrix, by replacing the values with row-wise mean values of original matrix. The covariance matrix of updated data is estimated using bias term M and matrix A will be computed using EVD. Principal components are calculated using Eq (7) (Ilin, Raiko 2010):

$$X = A^T \left(Y_c - M\right) , \quad (7)$$

where Y_c states centred principal component and M states row-wise mean values of original matrix.

For better estimation of missing values, the computation is reconstructed as in Eq (8) (Ilin, Raiko 2010):

$$Y_c = \begin{cases} Y \\ AX + M' \end{cases}, \quad (8)$$

where Y will be used for observed values and $AX + M$ for missing values.

In case of condition rating for a bridge element it is considered inappropriate to complete the dataset with algorithms, because unfilled rating usually indicates missing element (Hanley *et al.* 2016).

The suggestion has also been made for variables, which means PCA should only be conducted on continuous variables conforming to a Gaussian distribution (Qian *et al.* 1994), and its application to discrete data, such as element condition state ratings, are inaccurate. However, so long as inferential techniques requiring the assumption of multivariate normality are without reference, there is no necessity for the variables in the data set to have any associated probability distribution (Jackson 2003).

It is often considered wise to use the correlation matrix for a PCA, as the standardized varieties are dimensionless and more readily compared (Jolliffe 2002). However, when the variables are measured in the same units and have a low variance, using the covariance matrix is sometimes appropriate, and it is beneficial when statistical inference is essential. In case, when the condition ratings are already dimensionless, it is unnecessary to entirely standardise the variables (Hanley *et al.* 2015). Since condition ratings in Estonian BMS are already dimensionless, the raw data is used as input for a PCA.

4. Results

The Principal Component Analysis was conducted on five most common reinforced concrete bridge types shown in Table 2. In every specified typology, there was a different number of element groups present, and in some typologies, there were only a few records available on the condition of drainage, deformation joints or bearings, so these elements were excluded from analysis.

The comparison was made in two different steps. At first, two different algorithms of PCA were compared, concentrating on some useful PC and coefficients. Secondly, differences in PCA results between different typologies were compared and possible input for weighing factors suggested.

4.1. Comparison of algorithms

To compare different algorithms with identical data, suggested script by Matlab® was used. An example of results using ALS and SVD algorithms is presented in Fig. 2 and Table 4 where the mean values of condition states and included some observations of element groups are shown. As stated in the previous chapter, ALS algorithm fills missing data with row-mean values and input for calculating PCs are based on artificial data, which is different from SVD algorithm.

Differences in Fig. 2 are small when considering the condition state is within the limits of 1–4, but within the period of two different inspections, the condition state has changed in similar calibre, which means the decision-making process is influenced by artificial results.

In Table 4, only element group "Other" results have remained unchanged. The reason is the same amount of

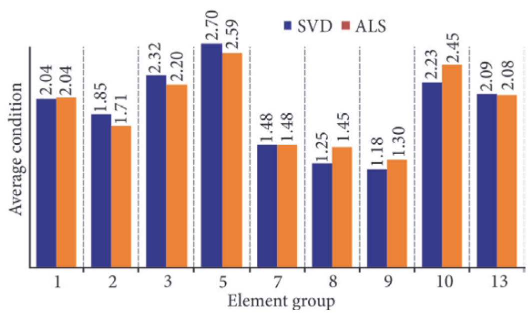

Fig. 2. Mean values of Group No. 2 (slab bridges) data of first inspection cycle

Table 4. Comparisonof different PCA algorithm results

Element group		Overlay	Barriers	Handrails	Slopes	Other	Waterproofing	Deck plate	Edge beam	Substructure
Singular Value Decomposition (SVD)	Mean	2.04	1.85	2.32	2.70	1.48	1.25	1.18	2.23	2.09
	Observations	177.00	63.00	79.00	86.00	265.00	90.00	90.00	89.00	177.00
Alternating Least Square (ALS)	Mean	2.04	1.71	2.20	2.59	1.48	1.45	1.30	2.45	2.08
	Observations	265.00	265.00	265.00	265.00	265.00	265.00	265.00	265.00	265.00
Alternating Least Square (ALS) difference in % from Singular Value Decomposition (SVD)	Mean	0.30	–7.60	–5.20	–4.10	0.00	15.60	9.80	9.70	–0.30
	Observations	49.70	320.60	235.40	208.10	0.00	194.40	194.40	197.80	49.70
Variance		0.46	0.66	0.76	0.46	0.47	0.33	0.03	0.16	0.14

observations in both algorithms, which in other element groups are filled with artificial data calculated with ALS algorithm. Differences in mean values of condition states differ from 7.60% to 15.60%, which is considered to be within the limits of human subjectivity, but still, have the additional influence on results. Differences in mean values and number of observations are directly uncorrelated, but elements with more missing fields have a more significant difference in mean condition states. On the other hand, when looking the ranking of element groups, the list remained unchanged.

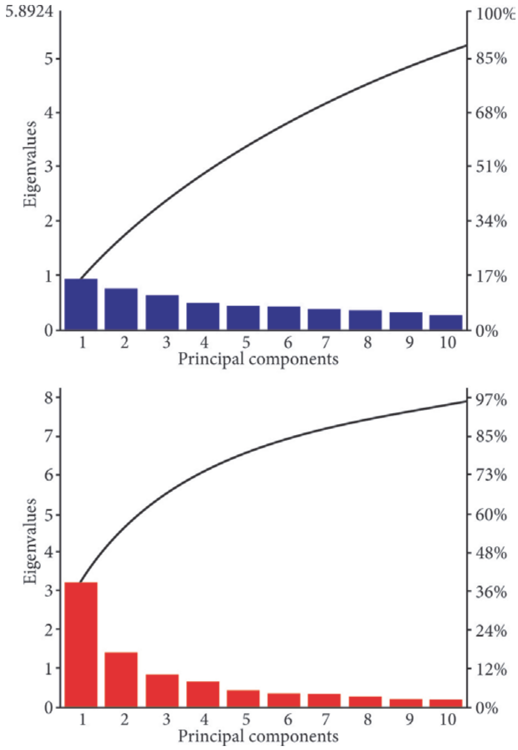

Fig. 3. Scree plot of Group No. 1 using SVD (left) and ALS (right) algorithm

Table 5. Conclusion of principal components and retained variation from different algorithms

Algorithm		Singular Value Decomposition (SVD)		Alternating Least Square (ALS)	
Group	Inspection cycle	Principal Components PCs	VAR retained	Principal Components PCs	VAR retained
1	1	6	63.8	4	73.7
	2	6	66.6	4	77.7
2	1	4	74.3	2	77.6
	2	5	79.2	2	74.5
3	1	5	82.7	3	74.6
	2	4	71.0	3	73.8
4	1	5	79.9	2	96.6
	2	4	77.3	2	97.2
5	1	4	83.0	2	71.9
	2	3	75.7	3	81.7

In conclusion, according to Hanley *et al*. (2016) arguments and overall results of the comparison, in asset management, it is correct to use data with original results to prevent using false information of elements, which are missing. Due to incorrect information, interventions must be carried out also on missing elements.

4.2. Number of principal components

The number of useful components was visualized with a scree plot of the eigenvalues (Cattell 1966). When it is necessary to determine, which PCs are essential and which must be discarded from the data set, then it is effective to use scree plot as a tool. As the components become less influential the slope of the scree plot begins to flatten because they have retained less variance than the previous components (Hanley *et al.* 2016). From the example of beam bridges shown in Fig. 3, the PC, at which the plot begins to flatten out, occurs for SVD at the fourth PC and ALS at the third PC. There is also a difference in the retained percentage of the variation in the data, being respectively 48% and 66% for different methods with substantial PCs. For Singular Value Decomposition, the retaining variation percentage is still under half of all data variation. Without artificial data, it is necessary to use results of six PCs to have a similar amount of variation described. In both cases, the inclusion of these PCs would not violate the practice of retaining eigenvalues higher than average value.

The comparison of different typologies, shown in Table 5, is made with the assumption where the variation of deterioration model is described by PCs, which have higher eigenvalues than average. Although in all cases it is possible to get minimal results with fewer PCs using ALS, some patterns are stressed out. Group No. 1 has 15 element groups represented, so the lack of principal components is understandable. In Group No. 5, there are only nine element groups represented and a subset of missing data is lower, which gives similar results from both algorithms. Differences and similarities in variables are described with a specialty of ALS algorithm, where missing values are filled with row-mean values of the original matrix, and as a result, the variation of less presented components are lower.

If ALS algorithm makes the whole database artificial, then it is necessary to make some changes in data collection during visual inspection of different bridges by levelling the amount of data in different groups. One possibility is to spread these groups based on the amount of data – one with more elements and other with less. Since ALS algorithm retains most of the variation in falce data, further results are based only on SVD algorithm.

The coefficients of Principal Component indicate the relationship between the bridge elements and principal component (Hanley *et al.* 2016). As results vary from positive to negative, the positive values indicate advanced damage for the element groups in the bridge, and a negative values indicates these element groups, which are in favourable conditions.

For first principal component the largest coefficients of PC 1 in every typology were for the non-structural elements indicating most advanced damage for types in handrails, for old slab typology in barriers and for frame typology in other elements. Situation is explained by the circumstance where most of bridges have old deteriorated handrails or barriers. As most of element groups with the largest coefficients in PC 1 are indicating elements with shorter life cycle, there is also more variation in visual inspection results. Unfortunately, these elements are non-structural and it are considered as irrelevant elements of a bridge, because they incorporate minimal risk to structural load carrying capability. Since the retained variation, percentage is relatively high in following components then it is necessary to include results from them.

For second principal component in most cases the largest coefficients were also for non-structural elements indicating most correlation to previous elements in slopes, deformation joints and waterproofing.

Since different typologies retain variation differently and do not have a correlation in respective principal components, then only example of one bridge typology is presented (Figs 4–5) and further discussion of possible weighing factors for every typology is presented in subchapter number 4.3.

Example of principal components is based on slab bridges Group No. 2, which represents the most common typology constructed before 1960's since the lengths needed to span were relatively low, and it was easier to cast in slabs than beams. The results are influenced by non-structural elements in every principal component. The background of the results is explained by adequate building quality of structural components, and naturally, non-structural elements deteriorate faster. The most variance is retained in safety barriers. The first structural component is waterproofing. Results of the same element group have different signs, for example, overlay, results in the first cycle show deterioration, but in second cycle it shows better condition. The reason is described with the situation, where on the first inspection, the attention was on other structural elements, and this element group was inspected superficially. The same pattern is applicable for other element groups.

4.3. Weight factors of element groups

According to Eq (3) and results of PCA, it is possible to obtain weighing factors for every specified bridge typology. Results were calculated with results combined with two inspection cycles, where squares of PCs were taken before averaging the results. In this way, different signs of coefficients were eliminated, and element groups with higher variation remained as principal elements.

Overall results are presented in Table 6. Results are similar for most of the typologies, where weight factors are higher in non-structural element groups, and only some basic element groups influence overall results.

Weighting factors are also representing the overall results of PC 1, which shows where the most variance of visual inspection data is retained in safety barriers,

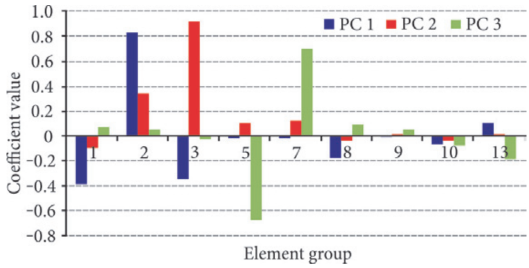

Fig. 4. Principal Component coefficients of first inspections of slab bridge typology

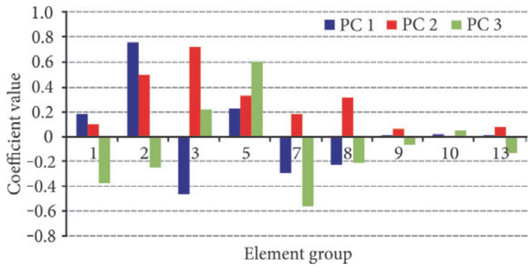

Fig. 5. Principal Component coefficients of second inspections of slab bridge typology

Table 6. Weighting factors λ of different bridge typologies

Element group	Typology Group No.				
	1	2	3	4	5
1	0.64%	8.84%	1.92%	2.92%	1.79%
2	2.08%	62.27%	7.70%	9.09%	5.09%
3	77.93%	16.95%	72.85%	56.68%	NA*
4	NA*	NA*	NA*	NA*	NA*
5	4.16%	2.51%	1.36%	8.10%	2.27%
6	4.70%	NA*	NA*	1.25%	NA*
7	0.34%	4.51%	6.26%	3.99%	86.08%
8	0.10%	4.15%	9.25%	2.07%	0.33%
9	0.03%	0.00%	0.27%	0.13%	0.01%
10	0.17%	0.20%	0.09%	0.36%	0.65%
11	0.10%	NA*	0.20%	0.29%	0.38%
12	0.24%	NA*	0.03%	NA*	NA*
13	0.31%	0.57%	0.06%	3.20%	3.39%
14	0.05%	NA*	NA*	0.04%	NA*
15	1.93%	NA*	NA*	5.52%	NA*
16	7.21%	NA*	NA*	6.34%	NA*

Note: *data not available

handrails or other elements. Three different reasons describe the background of these results. Firstly, there have been two different inspectors, each carrying out inspections in one year. Both inspectors were inexperienced at the beginning and gained knowledge during inspections. Secondly, these elements are widely damaged by traffic and snow ploughing. Also, the usage of de-icing salts makes steel elements deteriorate faster. Thirdly, the barriers and handrails are widely missing on older bridges, and due to safety restrictions, there have been recorded conditional state four of barriers instead of listing it as missing element.

Another critical finding in comparison of element groups among typologies there are differences in weight factors. It is incorrect to use same weighing factors for all the bridges in one network and even when overall typology is same, like Groups No. 2 and No. 3, the results are different.

The findings of this work should not be used directly in predictive models because the results are not taking into account the risk of failure or impact to load carrying capacity. One opportunity is to combine the results with initial weights or expert judgement using weighted PCA, but this topic needs additional investigation. On the other hand, the results endorse using PCA as a useful data reduction tool, because putting same weight factors to all structural components influences decision-making process.

5. Conclusions

There are three meaningful conclusions to be drawn from the statistical analysis of visual inspection data of reinforced concrete bridges in Estonian national roads.

1. In a comparison of different Principal Component Analysis algorithms, using Singular Value Decomposition algorithm is suggested because it uses the original data. Alternating Least Square algorithm results are useable with less principal components and with more specified coefficients. The main variation of Alternating Least Square results is hidden under missing elements, and condition states of element groups differ up to 15.6% from the original state and Singular Value Decomposition results. The difference in mean condition states of compared algorithms is increasing when the amount of missing information is higher.

2. Regarding different coefficients in first principal components, the main variation is retained in non-structural elements. According to this investigation, it is inappropriate to use Principal Component Analysis results directly in predictive models and for decision making it is important to consider additional circumstances as the risk of failure and influence to overall load capacity.

3. Reinforced concrete bridges in Estonia have similarities in principal components, but the importance of same elements are unequal. To be clear then although the main variance in Principal Components is retained in the same element, then according to results, different typologies retain variance in different elements, and due to fundamental distinction, it is incorrect to make decisions based on only construction material or typology.

A general conclusion is that Principal Component Analysis is suitable to be used as a statistical tool for data reduction and additional analysis of visual inspection data to filter out most significant components, but it is necessary to use additional weighing factors to emphasize the real influence of structural elements.

For future research, it is essential to cluster bridges based on a similar number of variables and add circumstances as additional weighting factors. It helps to provide relevant information without prioritizing common elements.

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Appendix II

Paper II

Kušar, M., Galvão, N., & Sein, S. (2019). "*Regular bridge inspection data improvement using non-destructive testing*." Life-Cycle Analysis and Assessment in Civil Engineering: Towards an Integrated Vision – Proceedings of the 6th International Symposium on Life-Cycle Civil Engineering, IALCCE 2018, 2019, pp. 1793–1797.

Regular bridge inspection data improvement using non-destructive testing

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ABSTRACT: Comparison of different bridge inspections shows that they vary greatly. Their implementation is dependent on the amount and type of data that has to be obtained for each individual bridge. Detailed inspection provides reliable data but it is costly, time consuming and not suitable for large scale use. Regular bridge inspection, on the other hand, is cost-effective, quick and, in general, the most suitable for conducting large numbers of annual inspections. However, data gathered is in most cases based on visual observation only, and as such, it is less reliable. In order to improve the quality of acquired data, certain non-destructive tests (NDTs) could be integrated into regular bridge inspection practice, while maintaining the above-mentioned advantages of visual inspection. Evaluation of NDT methods commonly used for bridge inspection is therefore performed. It is based on several relevant criteria: time consumption, procedure standardization, data reliability, diversity of use and complexity of results′ interpretation. The relative importance of the selected criteria is determined by experts from COST Action TU1406 by using Analytical Hierarchy Process. For each of the selected criteria, the assigned values are based on the literature survey. As a final step the utility function is defined, the NDTs with the same applicability level are compared, and their possibilities for use in regular bridge inspection are assessed.

1 INTRODUCTION

The condition of bridges within a road network has a direct impact upon the performance of the network as a whole and therefore, the bridges have to be regularly maintained. In order to optimize the maintenance strategy of existing bridge stock, their present condition needs to be determined first. The majority of data is obtained by conducting visual inspection (Kušar and Šelih, 2014); a method that may yield subjective and unreliable results. Nevertheless, this method will most likely remain the main aid for bridge assessment due to its simplicity and cost effectiveness. This point of view was accepted as an undisputed fact in WG3 meetings of COST Action TU1406 in April 2016 (Belgrade, Serbia) and October 2016 (Delft, Netherlands) (Kušar, 2017). In order to improve the quality of acquired data, selected non-destructive tests (NDTs) could be integrated into regular bridge inspection practice, while maintaining the major advantages of visual inspection – its time and cost effectiveness. The main advantage of NDTs is their repeatability without affecting/damaging the structure under investigation, but in order to be compatible with visual inspection, they must fulfill additional characteristics, such as results reliability, short investigation time and other characteristics discussed in this paper.

The main purpose of the presented research is to identify NDTs appropriate for use during regular bridge inspection. The paper begins with the selection of the criteria relevant for assessment of individual NDT suitability and determining the criteria relative importance. In the second step, the NDTs considered are presented, evaluated and classified with respect to their suitability for use.

2 IDENTIFICATION OF NDT METHODS APPROPRIATE FOR REGULAR BRIDGE INSPECTION PRACTICE

2.1 *Selection of criteria*

A variety of different criteria can be used to determine the NDTs suitable to be employed into regular bridge inspection practice. The selection of the criteria is based on the assessment of WG3 – Task group 1 and a review of similar research assignments (Gucunski et al. 2013, Hesse et al. 2015, Omar and Nedhi 2016) performed in the past. Depending on the method of determining the values, the criteria are classified as descriptive or measurable. For measurable criteria, a specific value can be determined (e.g. test duration time is 10 minutes, test is standardized). The values of descriptive criteria can be partially subjective as they cannot be precisely defined. However, based on expert judgement, they can be reliably classified into classes. Description of criteria selected is given in Table 1 and although some of the criteria selected seem to be related, they are independent. For example, standardization does not guarantee high reliability of results: pull off test is standardized (EN 1542) and its results are reliable, however rebound hammer test is also standardized (EN 12504-2), but the reliability of its results is questionable (Alwash et al. 2017). On site test duration and time required for the interpretation of the results are also independent: the duration of the pull-off test duration is relatively long as the adhesive between the disc and the substrate has to harden before the test can be executed, however test results are instant as no interpretation is needed. Usability and cost criteria are unrelated.

Table 1. Criteria to be used for NDTs assessment

Criteria	Description
Results' reliability	Descriptive criterion: It defines reliability or accuracy of the results/measurements. It deals with the technological perfection of the investigation (accuracy) and the sensitivity of the method to various external factors.
Standardization	Measurable criterion: If there is a standard prescribed for the NDT under consideration, the results should be more reliable.
Usability	Measurable criterion: It defines the number of parameters that can be measured with the NDT under consideration. Ability to investigate two or more materials, different types of damages or defects and similar.
Test duration	Measurable criterion: It defines the speed of the NDT execution and the speed of data acquisition. The criterion is predominantly related to the time spent by the inspector, but can also be related to possible traffic disruption (bridge or individual traffic lane closure) due to the investigation.
Results' interpretation complexity	Descriptive criterion: it relates to the obtained raw measurements and the need for long and demanding analysis to obtain final results (computer equipment and experienced engineers needed).
Cost	Measurable criterion: It defines the cost of equipment acquisition, the cost of test execution and the cost of data analysis.

2.2 *Determination of relative importance of criteria and evaluation*

In order to evaluate the NDTs under consideration, the relative importance of the selected criteria needs to be determined first. One of the most appropriate methods for group decision making, Analytic Hierarchy process (AHP), is used (Saaty, 1980). AHP has several advantages such as decomposition of decision problems into a hierarchy of more easily comprehended sub-problems that can be analysed independently and possibility of using the human factor as the basis for decision-making, in contrast to decision-making techniques that rely solely on the available data (Vodopivec et at. 2014). Its weakness is necessity to compare alternatives in pairs, with the number of required comparisons increasing rapidly. The disadvantage of the method may also be a lack of consistency in the judgements if the problem is not addressed properly.

In the survey presented, only engineers with experience in bridge inspection and NDT use participated, thereby minimizing the judgement subjectivity. Additionally, the consistency of judgements was increased using the SCB Associates Ltd software tool (SCB, 2017) that alerts the respondents in case of inconsistent judgements. The number of comparisons was limited by the relatively small number of criteria employed.

The comparison of criteria was conducted on a nine level descriptive scale (Table 2). Experts compared the selected criteria in pairs and gave their judgements. The judgements were considered satisfactory when inconsistency of the answers was below 10%. In case of larger inconsistencies, the experts were asked to re-evaluate their judgement in order to reach the required consistency. As the final step, the weight (relative importance) of each criterion was determined as the average value of all ratings attributed to the individual criterion by the experts. The results are shown in Table 3.

In addition to the selection of criteria, 'the criteria threshold values need to be determined (Table 4). Results' reliability is defined as the most important criterion. It is evaluated on the basis of the sensitivity of the test to external factors (e.g. humidity, temperature, test micro location) and on the basis of reliability of equipment or technology used. An example of NDT with high reliability is the phenolphthalein test as it provides reliable data regarding the carbonation depth at the micro-location. The rebound hammer, on the other hand, according to most researchers, is unreliable. To ensure higher reliability of rebound hammer test results, this method needs to be combined with other investigations or, alternatively, a large number of tests has to be performed (Alwash et al. 2017).

Table 2. Scale used in the pairwise comparison of criteria

Criterion descriptive value	Weight
Extremely less important	1/9
Very strongly less important	1/7
Strongly less important	1/5
Moderately less important	1/3
Equal importance	1
Moderately more important	3
Strongly more important	5
Very strongly more important	7
Extremely more important	9

Table 3. Criteria weights in order of importance

Criterion	Weight
Results′ reliability	0.280
Test duration	0.233
Results′ interpretation complexity	0.170
Cost	0.134
Usability	0.108
Standardization	0.075

Test duration is defined as the on-site time required for the test execution, while the time in the office is addressed by the result interpretation complexity criterion. The NDT is defined as a quick test if it can be carried out without substantially increasing the duration of the visual inspection of the bridge. Test duration is considered moderate if the time of the entire inspection is prolonged by up to 50% as a result of a NDT execution. If time consumption is greater, the usefulness of such NDT as a part of regular bridge inspection is questionable and should only be implemented when demonstrating exceptional performance in other criteria selected. For the test duration criterion, classification of the selected NDTs is to a certain extent subjective. Time needed to perform some tests is short, but these tests only provide local results and need to be performed numerous times to provide comprehensive results (e.g. hammer taping), while others require more time to be performed, but determine the state of construction as a whole for the parameter measured (e.g. infrared thermography). Additionally, for NDTs with wide usability, test duration may vary greatly for different measurements.

In addition to time spent for conducting field investigation, some NDTs require additional time for data interpretation that is usually performed in the office. Results interpretation complexity criterion considers NDT undemanding when its results are immediate (e.g. phenolphthalein test), satisfactory when short office or on sight analysis is required (e.g. cover meter requires undemanding data processing with computer software) and demanding when prolonged analysis with highly qualified personal is required (e.g. ground penetrating radar).

The cost criterion deals with equipment acquisition, maintenance, software cost and possible additional equipment needed for testing, while the value of inspectors' time is indirectly taken into account in the test duration and result interpretation criterion. The cost of acquisition is highly dependent on the technical characteristics of the equipment, therefore, the assessment based on this criterion is to some extent subjective.

Table 4: Criteria evaluation

Criteria	Scoring		
	3	2	1
Results′ reliability	High, external conditions do not affect the results	Moderate, various factors can affect the results	Low, complementary investigations needed to confirm the results
Test duration	Short, total bridge inspection time is not noticeably increased	Moderate, total bridge inspection time is prolonged	Long, total bridge inspection time is doubled
Results interpretation complexity	Immediate results	Short analysis required	Prolonged analysis and high professional qualification necessary
Cost	Low	Moderate	High
Usability	Investigation of various materials and their parameters possible	Investigation of one material and two of its parameters possible	Limited usability, only one parameter is investigated
Standardization	EN standard	National standard available	No relevant standard

NDT diverse usability is favourable for several reasons, such as less equipment needed on site and possibility of implementing unplanned types of measurements. The criterion is defined as less important as the inspectors are rarely surprised with the condition state of the bridge and consequently all tests are pre-determined. The standardization criterion was identified as the least important of the criteria selected. The NDT is given the highest score if it is supported by EN standard; if a national standard (domestic or foreign) is available a point is deducted and the lowest score is obtained if no relevant standard exists for the test under consideration.

2.3 *NDT selection and evaluation*

The most commonly used building material for existing bridges in Europe is, according to COST Action TU 1406 participants, reinforced concrete while the second and third most common materials are brick and stone. As the prevailing part of bridges in Europe are built from these materials, only NDTs suitable for inspection of these materials are considered in the paper.

The NDTs selected (Table 5) are scored according to each criterion described above. Scoring is based on literature dealing with various aspects of NDTs (SB-ICA 2007, Orbán and Gutermann 2009, Van der Wielen et al. 2010, Solla et al. 2012, Gucunski et al. 2013, Lee et al. 2014, Hesse et al. 2015, Hoła et al. 2015, Lee and Kalos 2015, Omar and Nehdi 2016, Rehman et al. 2016, Alwash et al. 2017, Omar et al. 2017) and COST Action TU1406 Work group 3 members' experience.

In some cases, the available data was contradictory, for example GPR is, according to SB-ICA (2007), costly and time consuming when larger areas are under investigation, while according to Omar (2017), the method is cost effective with an ability to rapidly scan large areas. Additionally, within the engineering field, there is a common belief that GPR raw data interpretation is very demanding; while Lee (2015) evaluated the method by the survey in which the participants assessed it as moderately difficult. In the literature review, several other contradictory views were recorded, making the evaluation difficult and open to different scoring.

The evaluation is performed based on scoring in Table 5. The utility function of an individual NDT (*Ui*) is determined as:

$$U_i = \sum_{c=1}^{m} V_{c,i} \cdot w_c \tag{1}$$

where i = NDT considered, c = criteria, $V_{c,i}$ = value of criteria c for NDT under consideration, w_c = criteria weight. NDTs are classified depending on their application into three categories: material properties, damage and defects and corrosion. Within the category, they are classified on the basis on the value of the utility function U_i (Table 6).

The results show that all NDTs measuring material properties have high utility rating. Despite some of the methods being semi-destructive (phenolphthalein, probe penetration and pull off test) and rebound hammer having poor reliability, these tests are fast, inexpensive and undemanding to perform, making them suitable for complementary use to visual observation during regular bridge inspections.

Table 5. NDT evaluation by the selected criteria

NDT	Scoring ($V_{c,i}$)					
	Results reliability	Test duration	Results int. complexity	Cost	Usability	Standardization
Cover measurement	3	3	3	3	1	2
Phenolphthalein test	3	2	3	3	1	3
Probe penetration test	2	3	3	3	1	2
Pull-off test	3	2	3	2	1	3
Rebound hammer	1	3	3	3	1	3
Impact echo	3	2	1	2	3	2
Thermography	2	2	1	2	3	1
Ground penetrating radar	2	2	1	1	3	2
Acoustic emission	2	2	1	2	2	2
Ultrasonic pulse echo	2	1	1	2	3	1
Half-cell potential	2	2	2	2	1	2
Galvanostatic pulse	2	2	2	2	1	1
Electrical resistivity	2	2	2	2	1	1
Linear polarization resistance	2	2	1	2	1	1

For damage and defects assessment, the NDTs considered are less suitable for use during regular bridge inspections. Their biggest weakness is high complexity of the results' interpretation, followed by the on-site test duration. As large number of bridges need to be inspected daily, time consumption is of utmost importance giving these tests lower utility. Impact echo investigation is rated visibly higher than other methods measuring damage and defects, and is at least conditionally suitable for use during regular bridge inspections. Further, according to SB-ICA (2007), crack measurements with impact echo method are quick and undemanding.

Based on literature review, all non-destructive methods dealing with corrosion detection and assessment exhibit similar characteristics (Table 5), consequently they all have similar utility function value. Their use in regular bridge inspection should be limited to bridges exhibiting high degree of corrosion only where reinforcement bars are already exposed, as most methods require direct contact with the reinforcement bars.

Table 6. NDT final classification

Application	U_i	NDT
Material properties	2,71	Cover measurement
	2,55	Phenolphthalein test
	2,43	Probe penetration test
	2,42	Pull-off test
	2,22	Rebound hammer
Damage and defects	2,22	Impact echo
	1,86	Thermography
	1,83	Acoustic emission
	1,80	Ground penetrating radar
	1,63	Ultrasonic pulse echo
Corrosion	1,89	Half-cell potential
	1,82	Galvanostatic pulse
	1,82	Electrical resistivity
	1,65	Linear polarization resistance

3 CONCLUSIONS

The paper discusses the possibilities of using NDTs for assessing the condition of reinforced concrete and masonry bridges as a part of regular bridge inspections. The survey of the methods considered shows that the methods measuring the material properties are best suited for complementary use with visual observation. The methods used to detect and measure damages and other irregularities are, in general, time consuming and their usefulness is largely conditioned by the time spent to conduct the investigations. Despite their relatively high reliability of results and diverse usability, they are currently only conditionally suitable for regular bridge inspection practice. Improvements that would allow faster test implementation or simplify the acquisition of results would significantly improve the applicability of these methods. All of the NDTs used to measure corrosion have similar characteristics and consequently the resulting values of the utility function are also similar. They are considered as less suitable for regular bridge inspection and their use should be limited to cases of bridges with a high degree of corrosion.

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Appendix III

Paper III
Sein, S., Matos, J. C., & Idnurm, J. (2019). "*Uncertainty in condition prediction of bridges based on assessment method – A case study in Estonia*". IABSE Symposium, Guimaraes 2019: Towards a Resilient Built Environment Risk and Asset Management – Report, 2019, pp. 1758–1765.

Uncertainty in condition prediction of bridges based on assessment method – case study in Estonia.

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Abstract

In this paper the uncertainty in condition assessment based on most common assessment methods, visual inspection and non-destructive testing, is investigated. For decision-making the averaged or estimated value is suitable, but if the basis of a decision is only a subjective visual inspection, then it could lead to a wrong decision. The second most traditional assessment method is non-destructive testing (NDT), which can give reliable results, but the interpretation of measurement is needed. To investigate the errors in both evaluations, benchmarking tests were carried out in Estonia within two groups, a group of experienced inspectors and a group of unexperienced students, to show how the importance of experience affects results. To present the influence of assessment uncertainty to condition prediction curves based on continuous-time Markov model are calculated and for updating, Bayesian inference procedure is used.

Keywords: reinforced concrete, non-destructive testing, visual inspections, assessment error, asset management, bridge assessment.

1. Introduction

The structural condition of bridges has a wide and direct impact for performance of the road network as a system. To keep the structural reliability high, bridges need to be regularly maintained. To optimize the maintenance strategies of existing bridge stock, their present condition needs to be assessed and determined. Based on COST Action TU1406 Working Group 1 Technical Report, most data are obtained by conducting visual inspection as an index form [1]. Visual inspection is a method that may yield subjective and unreliable results [2], but due to its simplicity and cost-effective data collection, this method will most likely remain the main aid for bridge assessment.

To improve the quality of acquired data, selected non-destructive tests (NDTs) are carried out additionally into regular bridge assessment practice. The NDTs are good for their repeatability without damaging the element under investigation, but to be compatible with visual inspection, they must be easy to use.

Unfortunately, all assessment systems are moreover database oriented and additional benefit can be added with Life Cycle Assessment by integrating deterioration models to predict the performance of these structures. Over the past twenty years, many models have been proposed including the ones-based on Markov chains [3], linear or non-linear probability functions [4], neural networks [5] and lifetime functions [6]. Although it is essential to obtain the assessment errors in the models, the amount of information makes the predictive models imprecise.

In this current work, Estonian bridge management system visual inspection methodology and most common standardized NDTs: sclerometer test carbonation depth and rebar depth, are tested by two groups with different expertise to clarify that visual inspections are unreliable method [7] for the bridge evaluation and more simple NDTs that are easy to carry out and not difficult to interpret. Results are then compared using Markov chain condition predictive models.

2. Visual inspections

Visual inspections in Estonia are carried out according to modified AASHTO methodology, where bridge evaluation is done on element basis. In this work, the bridge structure is divided into element groups that have been previously clustered by Sein et al. [8] as in Table 1.

Table 1. Classification of element groups [8]

Non-Structural Elements	Structural elements
Overlay	Deck plate
Barriers	Edge beam
Handrails	Piles and columns
Drainage	Supporting beam
Slopes	Wing wall, abutments
Deformation joints	Diaphragms
Other (river bed, signs etc.)	Main girder
Waterproofing	Bearings

Structural elements are subject of load carrying function to traffic and Non-Structural elements provide protection either to the structure or the users. The detailed list of bridge elements depends on the bridge structural type and is defined from pre-specified list of 145 elements during the bridge inspections.

Visual assessment of a bridge relies on inspecting every element unit of the bridge and evaluating each with a condition rating on scale from 1 to 4, based on the damage present and necessary rehabilitation method. Condition state 1 means, that element is in good condition and no rehabilitation is needed. Condition state 4 means, that element is in critical condition and major repair or replacement is needed. Condition states 2 and 3 describes the fair and bad condition respectively.

An overall element condition state is calculated based on the overall quantity of units and state factors [8] (1):

$$H_e = \frac{\sum_s k_s q_s}{\sum_s q_s} \cdot 100\%, \qquad (1)$$

where H_e – condition state of element; s is condition state; k_s – coefficient of state and q_s is the number of units in current state.

Condition Index (CI) is calculated like Health Index in Pontis [9]. The result is expressed with only one number between 0-100 and it is calculated based on element condition and weight factor. The CI shows the need for intervention [3] and it is agreed in Estonia, that an optimal level is reached, when bridge will be repaired before the CI is under 70. With CI less than 33, closing of the structure should be considered. In this current work, both element- and system level information is compared.

2.1 Inspections

The inspections were carried out on 3 common road bridge with similar structural typology, but conditions varied from good to almost critical. The overall information is presented in Table 2.

Table 2. Inventory information of visually inspected bridges

Bridge name	Lagedi	Assaku	Saku
Typology	Girder		
Length (m)	95.20	36.20	67.20
Spans	5	3	4
Material	Reinforced concrete		
Condition index	44	65	57
Year of Construction	1970	1989	1974
Year of repair	-	-	2005

The test inspection involved two groups of inspectors, where first group of 7 inspectors consisted of bridge experts with different background in bridge engineering and second group of 5 inspectors without any expertise.

In first group only 3 inspectors had previous expertise in the bridge assessment and familiar with the methodology, other inspectors had only read the inspection manual, but done similar bridge assessments previously. First group inspected two bridges, Lagedi and Assaku (Figure 1) viaduct.

Second group consisted of engineering Master students, who had no experience in bridge assessment. The methodology was introduced in one 1.5-hour lecture and they had read the inspection manual. Second group inspected Saku viaduct.

Both inspections were carried out in October and during all inspections it was rainy, which may influence the overall results and inspector motivation, but since all inspectors were in similar conditions, then it is considered that the weather condition didn't affect the differences of final evaluations.

2.2 Inspection results

The results are presented in Table 3 with every rating to bridge condition, overall mean and standard deviation (SD) of all assessments.

Table 3. Overall results of all inspections

Inspector	Lagedi CI	Assaku CI	Saku CI
1	48.9	66.9	49.3
2	36.7	66.7	74.0
3	38.2	66.8	19.3
4	36.7	87.0	76.1
5	46.0	75.1	70.1
6	39.8	87.0	-
7	55.3	88.3	-
Mean	43.1	76.8	57.8
SD	6.6	9,6	21,5

The overall mean of Lagedi viaduct is close to previous inspection result and SD shows, that 6 inspectors out of 7 assessed the bridge in a condition, where reconstruction is needed. Based on SD of results, the evaluations were in the same range. SD of assessment results of inspectors 2 and 3, who had experience more than 2 years, was 0.8.

Assaku bridge had a higher score for condiiton and overall inspection results were more scattered. The overall mean is more than 10 points higher than previous evaluation, but results are in similar range. Filtering out only experienced inspectors' results, then mean is almost the same as previous inspection result and SD of most experienced inspectors, 2 and 3, results was 0.1.

Saku viaduct was inspected by inexperienced inspectors and although the mean value is close to previous inspection result, only one inspector was close to the value. The SD of results shows, that results are scattered and using only one result can end with a wrong decision.

Further investigation is based on the element level data of Assaku viaduct. Three most experienced inspectors of first group, inspectors 1 to 3, have evaluated the CI between 66,7 to 66,9. The results are presented as maximum and minimum values, to show most extreme differences (Table 4).

Table 4. Extreme values of element level assessment

Element group	MAX (Inspector)	MIN (Inspector)
Overlay	66.7 (3)	60.5 (1)
Barriers	75.0 (3)	54.2 (1)
Handrails	66.7 (-)	66.7 (-)
Drainage	33.3 (3)	0.0 (2)
Slopes	64.1 (1)	40.3 (2)
Deformation joints	33.3 (3)	23.3 (2)
Other	100.0 (-)	100.0 (-)
Waterproofing	90.0 (3)	66.7 (2)
Deck plate	100.0 (2)	91.3 (1)
Edge beam	16.7 (1;2)	0.0 (3)
Piles and columns	66.7 (2;3)	62.5 (1)
Supporting beam	95.4 (2)	62.1 (1)
Abutments	66.7 (3)	59.3 (1)
Main girder	56.9 (1)	56.2 (2.3)
Bearings	91.1 (1)	55.2 (3)

Non-structural elements represent more elements with bigger area or quantity and due to that, the evaluations differ notably.

It is also interesting, that inspector number 3 tend have more maximum values and no minimal values, inspector number 2 have the opposite pattern.

Figure 1. Side view of Assaku viaduct (bms.teed.ee)

For structural elements, the most notable difference is the evaluation of bearings, which were easy to inspect on abutments, but without proper ladder it was not visible on piers (Figure 1). Bearings were the element group with maximum SD 16.7 between the results. The mean of element assessment ratings SD is 8.5, which is lower than SD of overall CI in first group.

3. Non-destructive testing

To increase the reliability of collected data, the visual assessment information should be updated with non-destructive testing information. During the comparison of different assessments, only few most common tests are investigated. Although two of the tests are standardized, the results of rebound test hammer have still questionable reliability [10]. All of tests are commonly used to detect material properties: compressive strength of concrete cover according to EVS-EN 12504-2, thickness of rebar cover and carbonation depth according to EVS-EN 14630.

All the tests are suitable only for reinforced or pre-stressed concrete structures, their test duration is short, and it is easy to obtain the results.

3.1 Bridges

Two reinforced concrete bridge were investigated and the main criteria for the selection was the casting technique of concrete: one should have vibrated and second not. Both selected bridges are common in Estonia. First tested bridge was Alliku bridge, constructed in 1975 and reconstructed in 2010. It is simply supported reinforced concrete slab bridge, it was renovated with strengthening of sub- and superstructure and concrete casting was done using nowadays methods.

The benchmarking test places were selected on two different elements: abutment and slab. Areas were marked and numbered.

The second bridge was Karutiigi, constructed in 1980, was simply supported reinforced concrete slab. It was reconstructed due to widening of the road in the beginning of 2018. Due to height of superstructure and flow rate of river, only one place was selected.

3.2 Inspectors and equipment

The benchmarking of NDT involved also two groups of inspectors like in visual assessment. First group, who carried out tests on both bridges, consisted of bridge experts with previous experience and knowledge in testing. Unfortunately, only 3 inspectors did the tests and there are only 2 different results to compare in each test. First group tests were done in period of May to June. Second group consisted of 8 engineering Master students, who had no previous experience in NDTs. Second group tested only Alliku bridge and tests were done in September. During both tests the temperature were over 15°C and weather was dry.

Sclerometer tests were carried out with SilverSchmidt Type-N concrete test hammer by Proceq. Values are presented as "Q"-value of impact instead of using conversion curves, which should consider the carbonation depth and will give rather conservative results for concrete strength. The thickness of rebar cover was measured with Proceq Profoscope+ cover meter and carbonation depth was measured using phenolphthalein solution and caliper. Values are presented as an average result.

3.3 Test results

Test results are presented in Table 6. Experts, marked as E, are numbered separately from students, who are marked as S.

In overall the rebound test hammer results are in one range and show small deviation. The second test area had smaller "Q"-value, but SD is 1.8. The third test area of Karutiigi bridge have only two different results with mean "Q"-value 49.7 and SD 0.8.

Table 6. Test results of Alliku bridge

Tester	Rebound value Q	Carbonation depth [mm]	Average cover depth [mm]
E 1	66.5	9	45
E 2	67.7	8	45
S 1	66.1	6	43
S 2	63.7	7	40
S 3	64.5	7	41
S 4	62.8	7	45
S 5	63.8	6	42
S 6	67.3	7	46
S 7	66.6	7	43
S 8	65.2	7	41
Mean	65.4	7.1	43.1
SD	1.6	0.83	2.0

The carbonation depth test results of area 2 show that experts obtained higher results compared to students although the results were in one range. In test area 1, the results were opposite – experts obtained smaller results than students. In both cases the SD is relatively high considering the relation to overall result.

The results are scattered because investigated test method requires decent cleaning of the drill hole and measurement should be taken within 30 seconds after the application of solution. Nevertheless, it is doubtful that with bigger carbonation depth the SD ratio to Mean value is similar.

Overall average results of concrete cover measurements are in one range (Table 6), but the average result is based on 5 to 10 measurements, which were in the range between 38 to 50 mm. In test area 2, the measurements were in smaller range and in test area 3 it was not possible to measure the cover depth, due to missing reinforcement.

In conclusion of NDT benchmarking tests, it is clear, that errors are smaller in comparison to visual inspections and non-experts can achieve higher accuracy without any previous experience in simple tests. It is still important to translate the results into quantitative scale like, for example, Mateus and Bragança [10] to start using these tests for regular assessment or as additional evaluation to visual assessment. In this research, only statistical errors and predictive model updating are investigated, but it for the future developments it is suggested to combine the mean values of NDTs with visual inspection outcome.

4. Predictive models and updating

For modelling, a probabilistic condition degradation model of an abutment of specific bridge is developed using Monte Carlo method. The model is based on historical information of Estonian national road bridges.

The Markov chains are stochastic processes, that are widely used for modelling information of existing bridges. Most extensively these models are based on a discrete scale, where transition between states is defined as (2) [11]:

$$\begin{bmatrix} C_1 \\ C_2 \\ \vdots \\ C_i \end{bmatrix}^T_{t+\Delta t} = \begin{bmatrix} C_1 \\ C_2 \\ \vdots \\ C_i \end{bmatrix}^T_{t} \times \begin{bmatrix} p_{11} & p_{12} & \cdots & p_{1j} \\ 0 & p_{22} & \cdots & p_{2j} \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & 0 & p_{ij} \end{bmatrix}_{\Delta t} \tag{2}$$

Where $C_{t+\Delta t}$and C_t are condition vectors at time $t + \Delta t$ and t, respectively. Vectors are defined as the probability of an element being in each performance state, C_j. Probability of transition between state i and j from instant t and $t + \Delta t$ is defined by p_{ij}, which is an element of a matrix P. If the intervals between inspections are not regular, as in Estonia, then continuous-time Markov process (CTMP) using transition intensity matrix have been proposed and defined as in (3) and (4) [12]:

$$\frac{\partial}{\partial t} P = P \times Q \tag{3}$$

$$P = e^{Q \times \Delta t} = \sum_{n=0}^{\infty} \frac{(Q \times \Delta t)^n}{n!} \tag{4}$$

Where, P is the transition matrix and Q is the intensity matrix, which represents the instantaneous probability of transition between the state i and j, where $j \neq i$. The intensity matrix for the deterioration process is present in (5) is calculated using state-dependent and time-independent model. Broader investigation of different CTMP formulations were done by Kallen and Noortwijk [12] using bridge condition data for statistical estimation.

$$Q = \begin{bmatrix} -\theta_1 & \theta_1 & 0 & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & -\theta_i & \theta_i \\ 0 & 0 & 0 & 0 \end{bmatrix} \tag{5}$$

Where θ_i is the instantaneous transition probability between adjacent state i and j. The initial estimate of matrix Q is calculated through (6) [13]:

$$\theta_i = q_{ij} = \frac{n_{ij}}{\sum \Delta t_i} \tag{6}$$

Where n_{ij} is the number of elements that moved from state i to state j, and $\sum \Delta t_i$ is the sum of intervals between observations.

The intensity matrix of abutment element group based on historical information Estonian bridges have following transition probabilities [0.0375;0.0230;0.0100].

Information is updated using Bayesian updating with informative prior. This approach has been introduced by Neves and Frangopol [3], by combining Bayesian updating with simulation for improving expert judgment. The obtained results showed significant impact on the prediction, when including the information obtained from inspections.

Based on the Bayes theorem, the probability density function of condition including the results of inspection can be defined as (7) [14]:

$$f''(C_T) = K \times L(C_T) \times f'(C_T) \tag{7}$$

Where $f''(C_T)$ is the probability density function of the condition at the time T considering both inputs, that are present in posterior distribution, $f'(C_T)$ is the probability density function of the condition at the time T considering only assessment, $L(C_T)$ is the likelihood function. K is a normalizing constant defined with (8) [14]:

$$K = \frac{1}{\int_{-\infty}^{\infty} L(C_T) \times f'(C_T) dC_T} \tag{8}$$

For the Monte-Carlo simulation, the mean and standard deviation of assessments were put into the scale of 1-4. The CI at time τ, can be calculated as (9), (10) [15], [16]:

$$\mu_C^\tau = \frac{\sum_{i=1}^{n} C_\tau^i \times L(C_T^i)}{\sum_{i=1}^{n} L(C_T^i)} \tag{9}$$

$$\sigma_C^\tau = \sqrt{\frac{\sum_{i=1}^{n} C_\tau^i \times L(C_T^i)}{\sum_{i=1}^{n} L(C_T^i)} - \left(\frac{\sum_{i=1}^{n} C_\tau^i \times L(C_T^i)}{\sum_{i=1}^{n} L(C_T^i)}\right)^2} \tag{10}$$

Where μ_C^τ and σ_C^τ are the mean and standard deviation of the CI at the time τ including model and assessment, C_τ^i is the CI at time τ connected to sample i, C_T^i C_τ^i is the CI at time T connected to sample i and n is the number of samples.

4.1 Results

For correct comparison, the standard deviation of different assessments is expressed as a ratio from the mean value. In addition, it is assumed that standard deviation of degradation model is 0.1. Highest standard deviation is from visual inspections carried out by master students, the standard deviation in the condition index scale is 1.49. Lowest standard deviation of visual inspection is 0.5, which is understandable, because if one evaluates element in condition 3, then the state can be between 2.5 to 3.5.

In comparison, the most scattered NDT results have scaled standard deviation 0.47. Most precise assessment results were obtained using rebound hammer, the scaled standard deviation was 0.10.

There are three aspects of differences that were compared: deviation in lowest assessment precision (Figure 2), best visual assessment (Figure 3) and highest assessment precision (Figure 4).

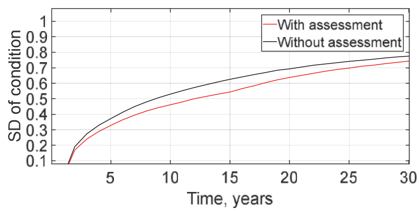

Figure 2. Standard deviation of condition with visual assessment carried out by inspector without any previous experience.

To present the influence, only one situation is visualized. The situation visualizes the change of standard deviation during 30 years of new element, which was put into operation in year 1, the assessment is made in year 15.

Even with lowest precision there is visible difference in two scenarios, which means that even without experienced inspectors, it is better for the owner to visually assess the bridges instead of using just degradation models.

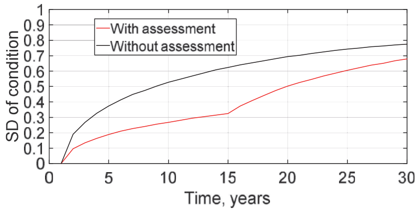

Figure 3. Standard deviation of condition with visual assessment carried out by inspector with previous experience.

In comparison of experienced and inexperienced inspectors, it is clear, that first ones can assess the elements more precisely. The difference of SD in year 15 is 0.22.

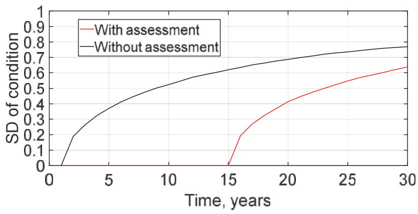

Figure 4. Standard deviation of condition with non-destructive assessment carried out by novice or expert.

Finally, in comparison of NDT and visual assessment it is also clear that in proper interpretation, the NDT is better option for precise assessment. In addition, for novice inspectors, it is more reliable to carry out tests than do visual inspection. Although, in case of testing, the places must be clarified previously.

5. Conclusions for discussion

The paper investigates the uncertainty of assessment carried out by people with different level of expertise, to present how the standard deviation of results influence condition prediction. The survey of visual assessments shows person without previous knowledge about bridge condition assessment can have almost three times higher deviation in their results than experienced ones. In visual assessment, inspectors with previous expertise can obtain results with small deviation in assessing the overall bridge condition, but element level assessment results have higher errors. In comparison of non-destructive testing methods three common and simple methods were compared. In case of test results, there are no clear difference between experienced and novice testers. In comparison of assessment methods, the most imprecise non-destructive test results are more accurate than visual assessment results.

For further investigation, it is suggested to put the non-destructive test results in similar scale to visual assessment results, add costs of assessment and analyze one method in simultaneous inspections to aggregate the number of assessments and costs during life-cycle of a bridge. In getting the most optimal condition control plan it is important to find the best combination of different methods to keep the standard deviation under desired level and minimize costs.

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Appendix IV

Paper IV

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BRIDGE MANAGEMENT

RC bridge management optimisation considering condition assessment uncertainties

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Abstract. Decision-making in bridge management has changed considerably in the past two decades and owners are additionally considering what types of interventions to implement, but correct decisions still need certain input. In Estonia, like in many countries, bridge management is based on inventory records and condition information. The main emphasis of this investigation is on improving the regular condition assessment. More accurate non-destructive testing methods and optimised inspection scheduling are proposed, to reduce condition assessment uncertainties. A conversion matrix for translating additional assessment results to the rating scale of the current Estonian Transport Administration management system is introduced and uncertainties in the condition state are analysed probabilistically. In addition, stochastic degradation models based on existing information are investigated to help considering uncertainties as a part of the overall management process. What impact the adopting of quantitative assessment, rather than qualitative visual inspection, may have on the suggested interventions schedule is also analysed. The probabilistic characteristics of the condition profiles of the most common bridge elements are computed using Markov Chain Monte Carlo stochastic simulation. The optimisation of inspection scheduling is performed considering the uncertainty of the initial deterioration model. When a threshold value, defined by the owner, is reached, the model is updated with assessment data to maintain the level of uncertainty below that threshold. The results confirm that deviations in the degradation model and assessment results influence the bridge condition uncertainty. Likewise, times of both inspection and intervention are influenced, which will ultimately impact the overall management reliability and costs.

Key words: bridge management, condition assessment, non-destructive testing, visual inspections, optimisation.

1. INTRODUCTION

Bridges are one of the most critical components of the transport network and they require regular investments to maximise economic and societal benefits. Efficient bridge management should meet the present and future needs of the users, normally under the pressure of limited funding. In the past three decades, these investments have been planned, managed, and technically supported by bridge management systems (BMS). BMS basic components are the following: inventory database, deterioration models, optimisation models and update functions (AASHTO 1993). Hence, the management process starts with a correctly formulated database, where relevant condition information is also stored.

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In most countries, the condition information is obtained from visual inspections (Kušar and Šelih 2014; Mirzaei et al. 2014), a method that is the most basic level in the hierarchy of structural assessment of existing structures (Rücker et al. 2006) and is based on qualitative expert judgement. It is a vague measure for the deviation of the inspected bridge from the “as new” condition as described by Hajdin et al. (2018). While this quick and cost-effective procedure, particularly when large bridge stocks are being managed, remains the main assessment method in the next years (Kušar et al. 2019), it has been criticised for being an unreliable method for the evaluation of the condition state of bridges (Phares et al. 2004).

The main source of uncertainty in visual inspections is related to its subjectivity, which means that different inspectors, under similar conditions, may evaluate differently the condition state of the bridge (Corotis et al. 2005; Kušar 2014; Sein et al. 2019). On the other hand, visual inspection outcome does not consider safety and serviceability aspects (Hajdin et al. 2018). The additional problem with the subjective visual inspection method is related to triggering maintenance actions without revealing information about the bridge’s inner structure. Thus, the results of inspections alone do not allow medium- or long-term planning and additional strategies must be developed (Neves, Frangopol and Cruz 2006; Neves, Frangopol and Petcherdchoo 2006; Taffe 2018). With qualified inspectors and proper guidelines, valuable information can be provided regarding construction methods, weathering, mechanical damage, deterioration, deficiencies, or other faults. To minimise the error arising from subjectivity, it is important to define standard assessment procedures (Rücker et al. 2006), as already done in a number of countries (Mirzaei et al. 2014). Moreover, throughout the past years, there have been numerous international projects concentrating on the development of advanced approaches for bridges both at the European level, such as quantifying the value of structural health monitoring (COST Action TU1402. 2015) or standardising the overall quality control of existing roadway structures (COST Action TU1406. 2015), and at the national level, projects like forming a bridge management system called the LeCIE_tool in Austria (Zambon et al. 2018).

To quantify the uncertainty of inspection, it is suggested that probabilistic values be used like the probability of detection, probability of false alarm (Rouhan and Schoefs 2003), probability of false indications (Straub and Faber 2003) or even the probability of good or wrong assessment (Sheils et al. 2012), with the main target to minimise the service life costs. Other approaches have tried to cover the inspection uncertainty related issues with overall asset management problems like rehabilitation timing (Zhang et al. 2008) or optimisation of lifecycle costs of bridge utilisation (Ghodoosi et al. 2018). All previous approaches can provide additional value to decision-making, but the methods are not causally linked to owners’ needs, e. g. the time between inspections or differences between assessment methods.

Taffe (2018) listed different methods for condition assessment and proposed a procedure of how data would meet the demands of the owner as customer. Although the work concentrated on the information of the inner structure, the issues regarding the definition of the measurand, identification of the method, location and timing of condition assessment are relevant. The proposed procedure targets accuracy of the results to guarantee their reliability, which means that precision should be ensured with the uncertainty of the measurement, which should be statistically evaluated using Guide to the Expression of Uncertainty in Measurement (GUM) (JCGM 2008) and the trueness must be provided by well-trained personnel (Taffe 2018). The main idea behind the quantification of knowledge is to identify the quantities influencing the results (Fig. 1) and allow to draw reliable conclusions (Taffe and Gehlen 2009), which should meet the minimum requirements of the client, not minimal in absolute (Taffe 2018). The framework was used in static calculations, but a similar approach and calculation methods can be utilised in the condition assessment of existing bridges because the outcome is also statistically evaluated.

The credibility of the measurand varies in correlation with the uncertainty of measurement, since the condition index is a value without an exact result and the result of the measurement is unknown. The uncertainty of measurement in the context of the condition index shows that the obtained value is better estimated with presently available knowledge (Hofer 2018). For the decision-maker, the effect of subjective visual inspection on the condition profile is especially important and may show high impact. Ilbeigi and Pawar (2020) developed a two-dimensional Markov process model that involved the current condition and the number of years an element has been in this condition. In combination with risk tolerance, they detected

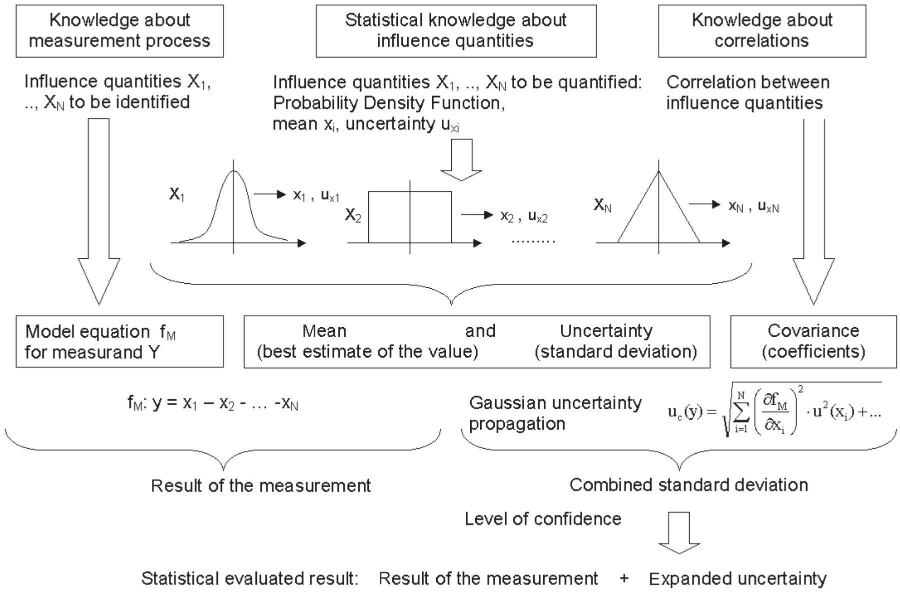

Fig. 1. Flowchart of knowledge about the measurement process and quantities influencing the outcome presented by Taffe and Gehlen (2009), based on GUM (JCGM 2008).

the optimal interval of visual inspections. The results indicated that the typical inspection interval was pessimistic and the optimal inspection interval for bridges in good condition could be 10 to 20 years, which is a notable difference from the common 4- or 5-year interval.

To improve the quality and reliability of collected information, appropriate quantitative and repeatable non-destructive testing (NDT) has been suggested (Kušar et al. 2019). Identification and selection of appropriate methods depend highly on the needs of the operator or owner, but overall, the accuracy of results needs to be assured, which, in turn, can only be estimated when they are correct and reasonably precise (Hässelbarth et al. 2006). Visual inspections are imprecise, and it is difficult to investigate the trueness of assessment even if the design information is available. Contrary to visual inspections, NDT has quantitative results, which makes it more precise, and if as-built information and environmental data are available, it is easier to assess the trueness of the results. In addition to trueness, there are environmental factors, age of different elements and deterioration rates of materials that introduce additional uncertainties that can be investigated using analytical models.

In the current research, the main measurand is the condition state of a specific bridge element group, but overall, the measurand can be any stochastic value representing the condition of a bridge or element. Since the true value of the condition is not known and the assessment of the accuracy of the best estimate is based on expert judgement, different assessment methods are compared within the context of the uncertainties, as well as time interval between inspections and interventions. The best estimate of the condition state is described by using a mean value, while the parameter of uncertainty is the standard deviation of the assessment method. Also, to combine NDT results with visual inspection data, a conversion matrix is proposed with clear threshold values helping the owner to translate the output to the condition. The degradation process is modelled using Markov processes combined with Bayesian updating considering the current condition with a probability distribution and the assessment result with standard deviation based on benchmark testing. Optimisation is performed by using one-dimensional interpolation of time with limit values for condition and uncertainty.

Although the examples are based on Estonian data, the outcome of this study intends to introduce uncertainty as a part of performance assessment, which helps to determine the inspection intervals and decide which type of assessment should be carried out.

2. PERFORMANCE ASSESSMENT IN ESTONIA

The investigated measurand, the condition state, is one of the performance indicators for the national bridge network of the Estonian Transport Administration (ETA), which helps to make decisions regarding intervention actions and restrictions to heavy vehicles. The input data has been collected from the ETA database, including registration and condition information for around 1000 bridges. Regarding condition assessment, the national regulatory documents state that the inspection interval should be three years (Minister of Economic Affairs and Infrastructure 2018) but since 2005 the interval of ETA bridge inspections has been four years. The overall bridge management process is not standardised and does not have clear manuals. It is mainly based on the PONTIS computer program with the numerical rating system (from 1 to 4) that uses the element inspection data (Roberts and Shepard 2000).

The performance assessment is mainly based on visual inspections performed with a 4-year interval. During the inspection, all element units of a bridge are assessed on a 4-level condition state, which means that one element can have many condition states. For example, a 10-metre-long beam can have 5 metres in the condition state “1” and 5 metres in the condition state “2”. The states are also related to intervention activities where the condition state “1” means that the element is as good as new and needs only maintenance, whereas “4” means that the elements are deteriorated and need to be replaced. All the elements are taken from a pre-specified list containing more than 100 inputs, which are like commonly recognised bridge elements introduced by AASHTO (Thompson and Shepard 2000).

At the bridge level, as well as at the network level, the main performance parameter is the Condition Index (CI), which is a number between 0 and 100, computed similarly to the Health Index (HI) used in most of the Departments of Transportations in the USA. The HI is defined as a normalised weighted average of the conditions of various elements, providing an overall indication of the health of the structure (AASHTO 2011), but for the CI the overall bridge index is calculated using element weight factors instead of the failure cost of the element. The index should still show the signs of deterioration unless adequate funding is obtained.

Although PONTIS included Markov chain-based degradation models and investment planning (Roberts and Shepard 2000), these modules have never been implemented in Estonian practice due to lack of relevant data preparation. Despite the first steps in the statistical analysis of the collected element information (Sein et al. 2017), using a multivariate methodology, the deterioration is still predicted with linear function or the annual average decay rate. However, before a substantive decision some non-destructive testing is also employed. This approach has been tolerated because in statistical representation the average bridge in Estonia is a small, simply supported beam structure built in 1974, made from reinforced concrete, has a median length of 14.2 metres, and annual investments related to bridge intervention activities have been around 5 to 8 million euros. Nevertheless, the introduced analysis shows clearly that there is room for improvement in performance assessment.

3. INVESTIGATION METHODOLOGY

The current research consists of three separate analyses, where the first two are related to real data. The first one is carried out to verify that the data-based degradation models of the historical condition are fit for purpose. In the second analysis, the differences in the interpretation of assessment results used for updating the model are presented and compared. Finally, the third analysis tests the application of optimal inspection scheduling based on the condition and uncertainty threshold.

The division is necessary because the condition information is estimated on real data, where data regarding all maintenance actions is missing and the results should be verified before using them in optimal inspection

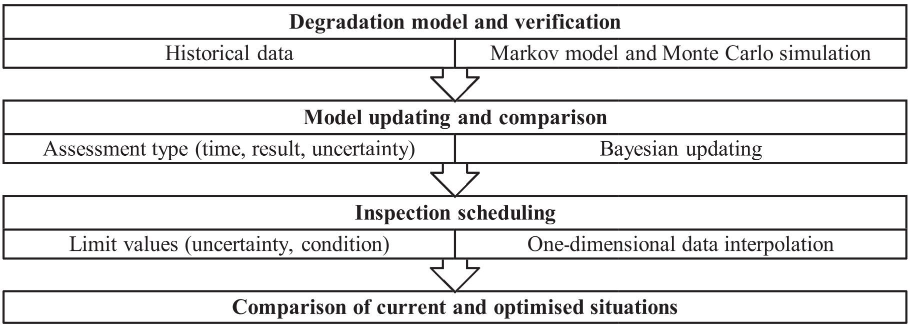

Fig. 2. Flowchart of the main elements of the proposed framework.

scheduling. In both of the first two analyses, the details from previously published studies were combined. The flowchart in Fig. 2 shows the different parts of the proposed framework.

The flowchart contains the main parts which can generally be divided into four, but the following steps are taken in more detail:

(1) Preparation of historical information to filter out typical bridges without registered interventions;
(2) Calculation of transition probability matrices for main element groups using the Markov model and Monte Carlo simulation;
(3) Verification of degradation models;
(4) Collection of additional information by means of inspections and non-destructive testing;
(5) Calculation of the condition with updated information;
(6) Comparison of model output based on the assessment method;
(7) Setting limits to the condition state;
(8) Determination of the maximal time frame to the next intervention action by combining the deterioration model and uncertainties in the model;
(9) Setting a limit to uncertainty in performance assessment;
(10) Determination of optimal inspection intervals to keep the uncertainty and condition state under the desired level.

4. DEGRADATION MODEL

The condition degradation model is a probabilistic model based on transition probabilities. In general, the values can be obtained either from accumulated condition data or by using an expert judgement elicitation procedure, which requires the participation of several experienced bridge engineers (Thompson and Shepard 2000). In this paper, the transition probabilities are obtained from accumulated condition data employing continuous-time Markov processes. The degradation models were calculated using a part of the software developed by Denysiuk et al. (2017), which were based on the details of Ferreira et al. (2014). These processes are used for two reasons (Kallen and van Noortwijk 2006):

(1) these models are suitable to implement when intervals between inspections are not regular;
(2) it is possible to include the uncertainty associated with irregular times between inspections in the model results.

5. FORMULATION OF THE MARKOV MODEL

A continuous Markov chain defines the condition state of an element in terms of a discrete variable and the transition between condition states is defined by an intensity matrix Q (Eq.1) (Jackson 2011):

$$Q = \begin{bmatrix} -\theta_1 & \theta_1 & 0 & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & -\theta_i & \theta_i \\ 0 & 0 & 0 & 0 \end{bmatrix}. \tag{1}$$

The intensity matrix Q defines the average instantaneous transition probability θ_i of moving from one condition state to another and allows that the transitions between the different states can occur on a continuous timescale. The initial estimation of transition probability from historical data is calculated as below (Eq. 2) (Jackson 2011):

$$\theta_i = q_{ij} = \frac{n_{ij}}{\sum \Delta t_i}, \tag{2}$$

where n_{ij} is the number of elements that moved from state i to state j and $\sum \Delta t_i$ is the sum of intervals between observations. In a continuous Markov chain, the transition probability P and the time Δt to move from one condition state to the next is characterised by the exponential distribution of the Chapman–Kolmogorov equation (Eq. 3) (Kallen and van Noortwijk 2006):

$$P = e^{Q \times \Delta t} = \sum_{n=0}^{\infty} \frac{(Q \times \Delta t)^n}{n!}. \tag{3}$$

To allow the consideration of uncertainties in the condition evaluation process, the final condition state profiles are computed using Monte Carlo simulation (Neves and Frangopol 2005) and the quality of fit is improved through an optimisation process by maximising the log-likelihood function (Eq. 4) (Denysiuk et al. 2017):

$$\sum_{m=1}^{M} \sum_{n=1}^{N} \log(p_{ij}), \tag{4}$$

where m is equal to the number of transitions observed in the element group, n is equal to the number of analysed elements and p_{ij} is the transition probability of the observed transitions predicted by the Markov model.

5.1. Verification of the degradation models

In the current investigation, the initial condition database consists of information for one of the most common bridge types, which is a simply supported reinforced concrete beam bridge with mostly precast elements. The elements are categorised differently from the condition state element classification into 16 groups including structural and non-structural elements (Table 1), using the classification of Sein et al. (2017). Since these groups had not been used in the system, the results of Markov models were verified with a goodness-of-fit test under the assumption that the goodness-of-fit follows a χ_n^2 distribution. In a typical deterioration model, only natural transitions from one condition to another are considered, and transitions that might occur because of maintenance actions are excluded. However, in Estonia the maintenance actions have not been recorded correctly and thus may affect the overall decay rate. The goodness-of-fit is measured by the discrepancy between the observed number of transitions and the expected number of transitions (Eq. 5):

$$T = \sum_{i=1}^{C} \frac{(O_i - E_i)^2}{E_i}, \tag{5}$$

where T is the goodness-of-fit metric, C denotes the number of possible condition state transitions (10 transitions), O_i refers to the observed number of transitions and E_i is the expected number of transitions of each time. The null hypothesis was assumed similarly to Ferreira et al. (2014), where Markov models were considered correct if the probability of goodness-of-fit is better than 5% of the sample value (Eq. 6):

$$P(T > T_{obs}) \geq 5\%. \tag{6}$$

The overall results are presented in Table 1. The limit value of χ_n^2 distribution for 5% significance level was calculated as below (Eq. 7):

$$T > \chi^2_{(11-1)-4:0.05} = 16.919. \tag{7}$$

Table 1. Goodness-of-fit test of deterioration models

Non-Structural elements	Observations	Discrepancy (T)	Structural elements	Observations	Discrepancy (T)
Overlay	222	1.87	Deck plate	253	1.34
Barriers	198	4.37	Edge beam	189	1.21
Handrails	169	0.69	Piles and columns	64	0.78
Drainage	43	0.13	Supporting beam	212	4.73
Slopes	231	2.31	Wing wall, abutments	203	2.42
Deformation joints	177	0.67	Diaphragms	155	0.85
Other (riverbed, signs, etc.)	155	29.79	Main girder	215	1.06
Waterproofing	253	16.75	Bearings	121	0.89

Based on the results presented in Table 1, only the non-structural element group designated "Other" did not pass the test, which means that there is adequacy between the sample and the model of all other element groups.

6. MODEL UPDATING

To include information from additional assessment, the initial deterioration model should be successively updated. The updating approach is based on the use of Bayesian updating combined with simulation and expert judgement proposed by Neves and Frangopol (2005). There are two differences in the current model compared to the initial work:

(1) the performance of structures was defined in terms of lifetime probabilistic condition, safety, and cost profiles. The condition index was combined with more consistent indicators such as the safety index, but in the current work only the condition index profile is used.

(2) the initial model did not include any information resulting from inspections or non-destructive tests, as it was based on the performance evolution over time obtained by using expert judgement alone. However, in the current analysis, additional information is included.

Similarly to the initial work, the mean, standard deviation, histograms, and percentiles of the life-cycle condition index are computed employing Monte Carlo simulation.

6.1. Formulation of the inference process

At the time of an inspection, the condition index can be characterised as a probabilistic variable, with a probability density function dependent on the results obtained by the inspector, but also on the quality of the inspection. Common practice defines the results of an inspection in terms of a set of possible outcomes (0, 1..., n). However, deterioration is considered a continuous or almost continuous evolution, and the results presented based on the simulation are a simplification of reality towards a pessimistic approach. In other words, this means that the real condition is always assumed to be worse than the average value. Based on the Bayes theorem, the probability density function of the condition including the results of the inspection can be defined as below (Eq. 8) (Neves and Frangopol 2008):

$$f''(C_T) = L(C_T) \times f'(C_T)\,, \tag{8}$$

where $f''(C_T)$ is the probability density function of the condition at the time T by considering both inputs that are present in posterior distribution, $f'(C_T)$ represents the probability density function of the condition at the time T by considering only assessment, $L(C_T)$ is the likelihood function.

For the Monte Carlo simulation, the mean and standard deviation of assessments at the time τ were calculated as below (Eq. 9) and (Eq. 10) (Neves and Frangopol 2008):

$$\mu_C^\tau = \frac{\sum_{i=1}^{n} C_\tau^i \times L\left(C_T^i\right)}{\sum_{i=1}^{n} L\left(C_T^i\right)}\,, \tag{9}$$

$$\sigma_C^\tau = \sqrt{\frac{\sum_{i=1}^{n} C_\tau^i \times L\left(C_T^i\right)}{\sum_{i=1}^{n} L\left(C_T^i\right)} - \left(\frac{\sum_{i=1}^{n} C_\tau^i \times L\left(C_T^i\right)}{\sum_{i=1}^{n} L\left(C_T^i\right)}\right)^2}\,, \tag{10}$$

where μ_C^τ and σ_C^τ are the mean and standard deviations of the condition at the time τ including the model and assessment, C_τ^i is the condition at the time τ connected to the sample i, C_τ^i is the condition at the time T connected to sample i and n is the number of samples.

6.2. Conversion matrix

Visual inspections of existing structures are a prime source of information in every assessment and a part of a management system. The observations can give reliable information on the structures and are normally integrated with prediction models for the assessment of deterioration in the infrastructure network. Nevertheless, using only visual inspections in decision-making makes the overall process less rational and more based on engineering judgement. While moving from prescription-based to performance-based quality control, the tendency to carry out additional non-destructive testing has increased. It can provide a more complete evaluation and might suggest maintaining the load-bearing components of the structure instead of more costly repair or reconstruction.

One reason why additional testing was not implemented in initial decision-making is related to the missing connection between different assessment results. For example, in Estonia the condition state description has an additional intervention suggestion, but NDT standards or manuals have only procedure descriptions and thus additional expert knowledge is needed. To enhance the use of NDT, the authors have compiled a table of suggested threshold values for some basic methods (Table 2).

The values in Table 2 are suggested by the authors based on the previous tests or research carried out in Estonia, the table is not complete and may be expanded with additional suitable NDT. The identification of criteria for potential NDT has been investigated by Kušar et al. (2019), where the suitable method was chosen based on the results' reliability, test duration, results' interpretation complexity, cost, usability, and standardisation.

Table 2. Conversion matrix of the NDT result to the condition state

Condition state, intervention recommendation	Carbonation depth [mm]	Chloride content [%]	Resistance of concrete (Andrade and Alonso 2004) [kOhmcm]
1 – very good, regular maintenance	Average carbonation depth is less than 25% of measured cover depth	Average measured value is less than 100% of the normative threshold value	Above 100
2 – good, local repairs	Average carbonation depth is between 25% and 50% of measured cover depth	Average measured value is between 101% to 150% of the normative threshold value	Between 50 and 100
3 – poor, overall repair	Average carbonation depth is between 50% and 100% of measured cover depth	Average measured value is between 151% to 200% of the normative threshold value	Between 10 and 49
4 – extremely poor, replacement or reconstruction	Average carbonation depth is more than measured cover depth	Average measured value is more than 200% of the normative threshold value	Less than 10

Table 3. Assessment results of typical bridges

Bridge No.	Construction year	Reconstruction year	Inspection-based		Carbonation depth		CS differences
			Time of inspection	CS – intervention	Average measurement, mm	CS – intervention	
883	1989	2002	2015	1.1 – M	6.9	1.4 – M	+0.3
826	1969	2010	2019	1.1 – M	18.7	2.3 – Rep	+1.2
907	1967	2001	2019	1.2 – M	20.0	2.8 – Rep	+1.6
911	1970	1998	2019	1.3 – M	28.0	2.8 – Rep	+1.5
908	1967	2001	2019	1.7 – M	13.6	2.6 – Rep	+0.9
503	1969	–	2019	2.1 – Rep	26.2	3.5 – Ren	+1.4
252	1965	2000	2018	2.3 – Rep	3.2	1.2 – M	–1.1
909	1974	–	2019	2.3 – Rep	37.5	3.5 – Ren	+1.2
306	1969	–	2019	3.0 – Ren	10.8	2.2 – Rep	–0.8

6.3. Assessment outcome comparison

To present the use of the conversion matrix and draw attention to the difference of the potential outcome as intervention activity for one specific element group, several bridges have been tested in the past years. The data presented in Table 3 were collected from the precast beams of nine bridges with the following common properties: made of reinforced concrete, simply supported beam bridges without additional diaphragms, constructed between 1965–1989 and designed according to the standard design of the Soviet Union, Catalogue No. 56-addition (USSR Mintransstroy 1962) or Catalogue No. 167 (USSR Mintransstroy 1963). The condition states of inspections (CS) are calculated based on the assessment result of element units and for carbonation depth, the result is interpolated. Possible intervention activities are regular maintenance (M), repair (Rep) or renovation/renewal (Ren).

It is possible to notice that only one bridge has the same potential intervention outcome, but in most cases, different assessment types have different outcomes. With two exceptions the inspection-based condition state is lower than the carbonation depth-based condition state, which means that the element should be repaired but is maintained instead (4 out of 9) or the element should have already been replaced or renovated but is repaired (2 out of 9).

The obtained results justify the question about the knowledge of uncertainties in the interpretation of data, because using only one assessment result as a trigger for intervention may lead to inefficient management. To consider the different assessment results, a confidence level of 95% is used in finding the optimal inspection interval and the inspection result is assumed to be the same as the average degradation model value. To improve the quality of data interpretation, it is suggested that the stochastic model should be combined with analytical models. For example, Zambon et al. (2019) has presented a framework where the analytical model of carbonation was combined with a Markov chain model.

7. UNCERTAINTIES IN PERFORMANCE ASSESSMENT

Apart from the non-trivial combination of different methods, also the uncertainty of assessment plays an important role in more accurate results and in moving towards optimal inspection intervals. When the value of a bridge condition is reported, in addition to the best estimate of its value, the best evaluation of the associated uncertainty should also be given, as it is not normally possible to decide in which direction the realistic condition of the bridge element is from the assessed condition and whether it performs as intended.

If the uncertainties are understated, then too much trust might be put in the values reported, which may lead to undesired consequences. Likewise, the overstatement of uncertainties could also have undesirable repercussions. For example, it could cause unnecessary interventions, making structure maintenance more costly.

Uncertainty, namely epistemic uncertainty, originates from various sources, and based on the classification of Regan et al. (2002), it can be divided into six classes:

- Inherent randomness, which is the uncertainty related to the randomness of the inherent nature affecting the outcome. This type of uncertainty can be quantified using probabilistic models;
- Measurement error, which is the uncertainty related to measured quantity and errors. This type of uncertainty can be quantified using probabilistic models if the measurement error is estimated;
- Systematic error, which is the uncertainty related to the bias of measurements or sampling. This type of error can be quantified but is difficult to notice and use in probabilistic models;
- Natural variation, which is the uncertainty related to natural conditions. Since the changes in natural conditions are unknown, careful consideration is needed before quantification of this type of uncertainty;
- Model uncertainty, which is the uncertainty related to the model abstraction of the real process. This type of uncertainty also needs careful consideration before quantification. Additionally, cause-and-effect relationships are exceedingly difficult to quantify;
- Subjective judgement, which is the uncertainty related to the interpretation of data. This type of uncertainty is difficult to quantify, similarly to model uncertainty.

In bridge management, all the described types of uncertainty are present, and it is impossible to separate them from each other. To help the bridge owners to make justified decisions and be aware of various sources of uncertainty, it is proposed that a limit to expanded uncertainty should be obtained which defines an interval with a specified level of confidence, and which satisfies their needs. However, the Joint Committee for Guides in Metrology (JCGM) suggests in their guidance (Bich et al. 2006) that such factors must be applied to the uncertainty as determined by a realistic method, where the uncertainty has been determined, the interval defined by the expanded uncertainty has the level of confidence required and the operation may be easily reversed. There are three distinct advantages of adopting an interpretation of probability based on the degree of belief, standard deviation, and the law of propagation of uncertainty as the basis for evaluating and expressing the uncertainty in measurement (JCGM 2008):

- The law of propagation of uncertainty allows the combined standard uncertainty of one result to be readily incorporated in the evaluation of the combined standard uncertainty of another result in which the first is used;
- The combined standard uncertainty can serve as the basis for calculating intervals that correspond in a realistic way to their required levels of confidence;
- It is unnecessary to classify components as “random” or “systematic” (or in any other manner) when evaluating uncertainty because all components of uncertainty are treated in the same way.

Although the JCGM (2008) guide states that when the standard uncertainty of an input quantity cannot be evaluated by an analysis of the results of an adequate number of repeated observations (stated as Type A evaluation), a probability distribution must be adopted based on knowledge or expert judgement (stated as Type B evaluation) which is much less extensive than might be desirable. That does not, however, make the distribution invalid or unreal; like all probability distributions, it is an expression of what knowledge exists. Therefore, measurement-based evaluations of standard uncertainty are not necessarily more reliable than knowledge-based evaluations (JCGM 2008). In the current framework, both types of evaluations are possible to use, but to increase the reliability of results, a comparison between uncertainties obtained by different test methods, with multiple benchmarking tests involving experts and novice users, was carried out by Sein et al. (2019). Based on those results, the measurement-based uncertainty is expressed as the coefficient of variation in Eq. 11:

$$CoV = \frac{\sigma}{\mu}, \tag{11}$$

where σ is the standard deviation, μ denotes the mean of the measurement and CoV is the ratio or relative standard deviation. Inspection related uncertainty values are based on benchmarking and presented as a

Table 4. Overview of uncertainties of different assessment methods (Sein et al. 2019)

Method	Expertise	Standard deviation ($CoV \times 4$)
	Inexperienced students	1.49
Visual inspection	Engineers	0.61
	Experts	0.50
Carbonation depth with phenolphthalein method	Inexperienced students	0.47
	Experts	0.35
Cover depth using electromagnetic cover meter	All	0.19

standard deviation of condition assessment (Table 4). The same values are used in the optimal inspection scheduling.

Inspectors with previous expertise can obtain condition state results with lower standard deviation than less experienced ones, but more accurate NDT methods give better results, even for inexperienced users.

If the assessment is combined, then the combined standard uncertainty is calculated by Eq. 12 and used in the likelihood function instead of the standard deviation of a single assessment:

$$u_c = \sqrt{((u)_1)^2 + ((u)_2)^2 \dots (+(u)_n)^2} \ , \tag{12}$$

where u_c is the combined uncertainty and $(u)_{1,2\dots n}$ are the standard deviations of separate assessments. For example, when combining cover depth measurement with carbonation depth measurement carried out by experts, the combined standard uncertainty is 0.40.

8. INSPECTION SCHEDULING

The main goal of the inspection scheduling is to keep the level of uncertainty under the desired threshold value by maximising the time between inspections. Based on the investigation of benchmark tests in Estonia (Sein et al. 2019), the inspections are an important part of the bridge management process and even assessment performed by an inexperienced inspector decreases the level of uncertainty in condition prediction. Unfortunately, the visual inspections have limitations due to their subjectivity and to increase the quality of assessment, it is necessary to combine the evaluation with quantitative measurements.

Finding the optimal solution is based on linear interpolation and the outcome is the time of inspection or intervention. The main goal of the optimisation is to maximise the time between inspections while keeping the level of uncertainty under a specified threshold value. As an additional result, the potential intervention can be triggered if the knowledge of the measurand minimises the costs of overall management processes.

The optimisation is based on the linear interpolation of the condition profile with 95% confidence. Linear interpolation can be expressed in general (Eq. 13):

$$y = y_0 + \frac{(x-x_0)(y_1-y_0)}{x_1-x_0} \ , \tag{13}$$

where $x_{0,1}$, $y_{0,1}$ are the coordinates of two known points and x, y are the coordinates of the unknown point. The known points are the time and error of predicted or updated conditions and the unknown point is partially defined with the limit value. Limit values for the condition state and uncertainty in the model are related to owners' policies or needs and need to be defined separately. The limit value for the condition state is one main trigger for potential intervention activity or in-depth assessment and the limit value for uncertainty is one main trigger for inspection but can also be used for triggering in-depth assessment or preventive intervention.

To investigate which of the proposed inspection scheduling gives optimal output, two parameters are compared. For uncertainty, a trapezoidal numerical integration is used to calculate the approximate area of uncertainties during the designed service life of 100 years. To simulate costs for the agency, total costs of element inspection and potential intervention are summarised.

9. CASE STUDY

Case study analysis concentrates on the same precast beams of the most common bridges as introduced in Chapter 6.3. The input is based on real values, but the investigation of different assessment methods is theoretical. Different inspection scheduling and assessment methods are compared during the service life of 100 years using the condition limit for triggering intervention. The emphasis is put on the total number of inspections, intervals, overall costs, area of uncertainty and the years when uncertainty is above a threshold value.

The limit values used for the condition and error are set based on the current maintenance policy of the ETA and expert judgement under the assumption that both variables follow a normal distribution and have a confidence level of 95%. Hence, the following restraints are applied:

- Condition state is limited to 3.0 (poor) because the elements look visually bad and most likely a renewal will be triggered within the next few years. Since the investigation is concentrated on inspection interval scheduling, only the decrease in the condition state is considered the only effect of maintenance action. As stated before, then the regular maintenance is included in the initial condition degradation profile.
- Uncertainty of a condition state is only limited to 1.0 because this gives the confidence that the assessed condition stays within the limits of one value. Additionally, currently used non-destructive testing methods are not fully accurate and limiting the uncertainty to lower values triggers inspections every year.
- The initial condition and uncertainties are included in the initial probability distribution, which is based on the expert judgement. For example, Kang and Adams (2010) used random errors of ± 10%, ± 20% and ± 50% in their sensitivity analysis of bridge HI related element condition assessment. In this investigation, the initial condition accuracy is ± 10%, which means that if the initial condition state is one, then the vector of probability distribution is *P(0)=[0.90 0.10 0 0]*.
- The standard deviation of the degradation model $\sigma_{model} = 0.50$.
- For regular visual inspection with σ = 0.50, a price of 150 EUR is estimated; for cover and carbonation depth measurements with σ = 0.40, a price of 400 EUR is estimated; for advanced NDT assessment including chloride content and resistivity measurements having σ = 0.3, the price is estimated to be 800 EUR.
- Intervention activity is the renewal of a beam during an overall bridge reconstruction and the cost is estimated to be 10 000 EUR.

Since the analysis is based on a specific bridge element group, the verified transition intensity matrix is used (Eq. 14).

$$Q_{beams} = \begin{bmatrix} -0.0100 & 0.0100 & 0 & 0 \\ 0 & -0.0237 & 0.0237 & 0 \\ 0 & 0 & -0.0137 & 0.0137 \\ 0 & 0 & 0 & 0 \end{bmatrix}. \quad (14)$$

The transition probabilities are low because there are only three different visual inspection inputs since 2005 and many new structures are still evaluated in the condition state 1. Based on the presented values, an average condition degradation profile is calculated. Further analyses are carried out using the probabilistic degradation model with a confidence level of 95% and the previously stated restrictions (Fig. 3).

Due to the low transition probabilities, the average degradation profile is almost linear, which makes the scheduling of intervention activities unrealistic. Nevertheless, considering the errors of the model with 95% confidence, the renewal of concrete beams is triggered after 43 years in service. In Fig. 4, the probability mass distribution of the degradation model in year 42 is presented, the initial standard deviation has increased from 0.30 to 0.85. From an optimistic point of view, it means that there is a possibility that the real condition state is still 1.0, but intervention activity is nonetheless triggered. In comparison, taking into consideration that the inspected condition affects the final condition probability mass function, inspection with the result equal to 1.60 is presented.

It is clear that the model with updated information has a lower level of uncertainty and if the result of the assessment is true, then intervention activity can also be postponed. Currently, the condition assessment is

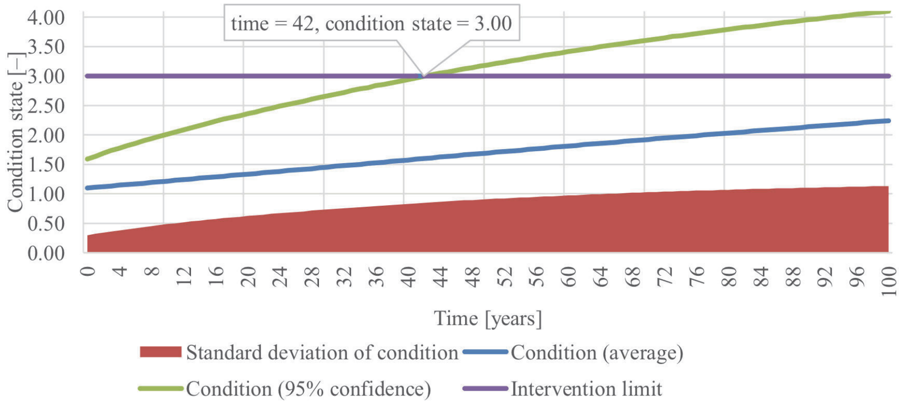

Fig. 3. Condition degradation profile of concrete beams.

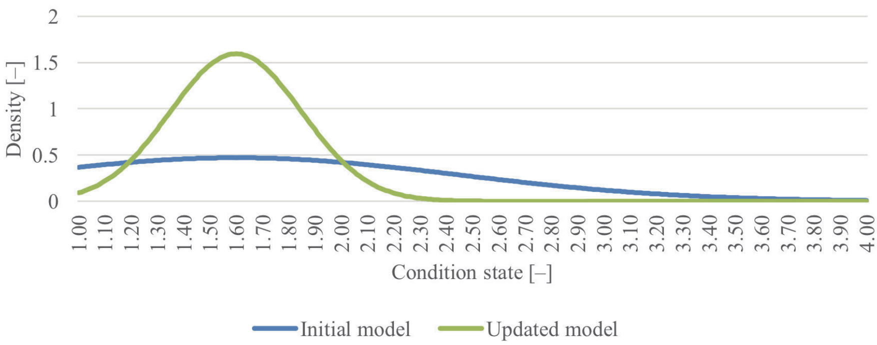

Fig. 4. Probability mass function of the initial and updated models on year 42.

only visual inspection based, with 4-year intervals. Since the cost of an inspection is low, it is still possible to make the current assessment regulation more optimal with clearer inspection scheduling. The average 4-year inspection interval is compared with the optimal inspection interval and the condition profiles are presented in Fig. 5.

Comparing the 4-year interval profile to the initial condition degradation profile in Fig. 3, it is possible to postpone the intervention by 41 years to year 82 with only better knowledge about the condition state. The limit of uncertainty in the optimal inspection interval triggers intervention in year 69, which is earlier than the current system, but without more accurate inspections the intervention is needed. If the trigger were based on the condition state, then the time of intervention would be in year 76. It is possible to see that before the intervention there are six inspections with an annual interval and this situation could be avoided with a more accurate assessment with the standard deviation $\sigma = 0.3$ as presented in Fig. 6.

Although accurate assessment is more expensive, it reduces the uncertainty and extends the time of the next inspection from 3 years to 6 years. Since the level of uncertainty is still high because of the uncertainties in the model, next inspections should also be performed with additional testing, and eventually it is not possible to postpone the intervention with a more accurate assessment.

Alternatively, it is possible to add cover depth and carbonation measurements to all inspections, with this addition it is possible to decrease the combined standard deviation to 0.40. The cost of the intervention will

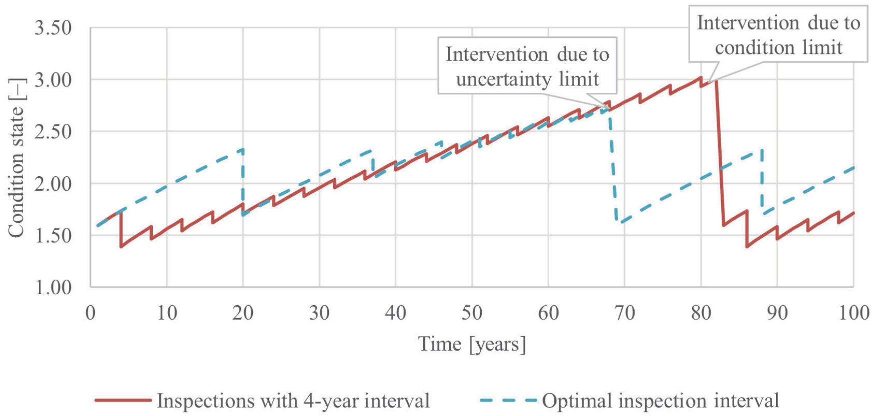

Fig. 5. Condition profiles of 4-year and optimal interval visual inspections.

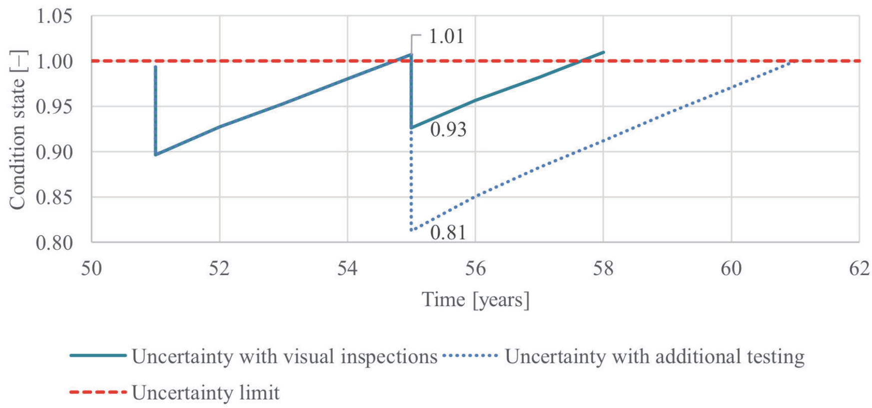

Fig. 6. Uncertainties in year 55 with different assessments.

be higher, but in return, the intervention will be triggered later. As in Fig. 5, the average 4-year inspection interval is compared with the optimal inspection interval. The condition profiles are presented in Fig. 7.

In both cases, the intervention is triggered in year 90 due to the condition limit. The main difference between the two approaches is the time between inspections, which is longer with optimal scheduling, as well as the number of inspections. With a 4-year interval, there were 22 inspections, but with the optimised approach, there were only 12 inspections with the intervals ranging from 5 to 19 years. Inspection intervals triggered by the uncertainty limit for visual inspections presented in Fig. 5 varied from 1 year to 20 years, which interestingly corresponds to the results based on optimising the inspection interval for New York bridges (Ilbeigi and Pawar 2020).

In the context of overall management, the overall costs should also be minimised, because more expensive and accurate tests can postpone more costly intervention. However, the situation where the assessment costs are higher than the intervention should be avoided. Assessment methods were compared during the service life of 100 years and an overview of the results is shown in Table 5.

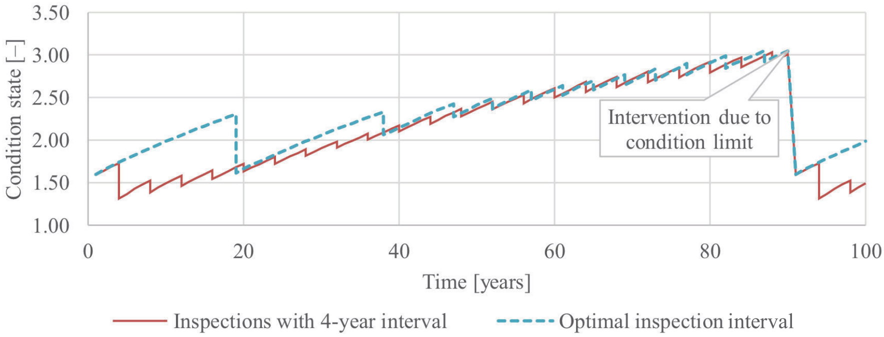

Fig. 7. Condition profiles of 4-year and optimal interval inspections with additional testing.

Table 5. Overview of total costs and uncertainties with different assessment approaches

Assessment type	No. inspections	4-year σ = 0.5	Optimal σ = 0.5	4-year σ = 0.4	Optimal σ = 0.4	4-year σ = 0.3	Optimal σ = 0.3
Total number of inspections [–]	0	24	14	24	12	24	7
Total number of interventions [–]	2	1	1	1	1	0	1
Total cost [EUR]	20 000	13 600	12 100	19 600	14 800	19 200	15 600
Area of uncertainty curve [–]	141.5	75.5	82.4	73.2	84.0	68.7	80.1
Uncertainty above 1.0 [years]	81	21	1	0	0	0	0

Comparing the triggered interventions, only the most accurate 4-year interval inspections can lead to a situation where no intervention is needed since the intervention is triggered due to the condition in year 103. Contrarily, the approach with no inspections triggered two interventions. The total cost is only 4.17% higher without any inspections. The lowest total cost results from visual inspections with the optimal interval. Most commonly one intervention is triggered, which means that the cost of intervention affects most of the approaches with the same amount. Although the area of the uncertainty of the optimal approaches is always higher, the uncertainty of the condition is always below the threshold value 1.0, which means that inspections are triggered only when needed and not based on a strict schedule. In the current 4-year interval approach, the inspections are triggered too quickly in the first 55 years and after that the level of uncertainty is higher than the threshold value, meaning that the condition could differ more than one state and intervention or more accurate testing is needed. Improving visual inspections by means of more accurate NDT is also justified because the uncertainty is lower than the threshold value, but the area of uncertainty is higher. Also, after 55 years the time between inspections can only be extended with better knowledge.

10. CONCLUSIONS

This paper has focused on intervention-related decision-making and inspection scheduling triggered by uncertainties in regular performance assessment. To integrate more accurate non-destructive testing methods with the currently common visual assessment, a condition conversion matrix and an overview of carried out tests has been presented. The different assessment method of the existing structure will probably lead to different intervention activity. Moreover, the data were theoretically analysed as probabilistic values of the beam condition of the most common bridge type in Estonian national roads. For computation, a Markov Chain Monte Carlo simulation was employed and verified with the Estonian Transport Administration's database.

Using one-dimensional interpolation of the uncertainty of the deterioration model, it was detected that, although the overall level of uncertainty is lower, the current condition scheduling policy is too pessimistic in the first 55 years after construction and too optimistic afterwards. This finding correlates with suggestions given by Ilbeigi and Pawar (2020) based on the New York bridge network inspection scheduling. The main conclusions about the knowledge of the uncertainty regarding the assessment are as follows:

- Probabilistic expression of assessment results helps to numerically present the knowledge about measurement process, prediction and uncertainties related to the result. Additionally, results that are presented based on the simulation of the degradation process are a simplification of reality, which means that the assessed condition must decrease the uncertainty. Using the condition profile of 95% confidence level instead of the average condition profile in a situation where transition probabilities are low is justified as this helps to keep the structure on the safe side by triggering interventions based on the condition state limit.
- Inspections help to reduce the uncertainty and generally postpone the time for intervention. Adding a limit to the uncertainty of the condition can cause a situation where intervention is triggered due to the uncertainty limit.
- Non-destructive testing methods are more accurate than visual inspections and are useful in extending the time interval between inspections due to a higher reduction rate of uncertainties.
- In addition to a reduction of uncertainty, improved assessment can have a different intervention outcome and the differences in the average condition state of visual assessment and common non-destructive tests can vary from –20% (–0.8) to +40% (+1.6).
- Compared to the 4-year inspection interval, optimal inspection scheduling keeps the level of uncertainty under the desired threshold value. Additional assessment based on cover and carbonation depth measurements is justified because the area of uncertainty is maximised and after 55 years, the time between inspections can only be extended with a better assessment.

Considering all the factors, it can be concluded that with simple improvements in assessment methodology it is possible to reduce the level of uncertainty in decision-making regarding the intervention activities of existing structures.

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Sildade haldamise optimeerimine lähtuvalt seisukorra hindamisega kaasnevast määramatusest

Sander Sein, Jose Campos Matos, Juhan Idnurm, Martti Kiisa ja Mário Coelho

Sildade haldamine on viimase paarikümne aastaga märkimisväärselt muutunud, sest oma otsuste tegemisel arvestavad omanikud lisaks seisukorra infole ka alternatiivsete parendustegevustega. Vaatamata muutunud olukorrale vajavad õiged otsused piisavalt täpset sisendit, mis jätkuvalt põhinevad registri- ja seisukorrainfol. Antud töös on keskendutud seisukorra hindamisele, tutvustades võimalusi selle edasiarendamiseks täpsemate mittepurustavate katsemetoodikatega ja analüüsides eri võimalusi, muutmaks hindamistevaheline aeg optimaalsemaks, tagades samal ajal, et otsuse aluseks oleva prognoosimudeli määramatus jääb alla piirväärtuse. Täpsemate mittepurustavate katsemetoodikate hõlpsamaks kasutuselevõtuks on autorid välja pakkunud üleminekumaatriksi, mis on koostatud lähtuvalt Transpordiameti praegusest kvalitatiivsel visuaalsel ülevaatusel põhinevast hindamissüsteemist. Töös on analüüsitud kolme erinevat seisukorra hindamisega kaasneva määramatuse aspekti, mis kõik põhinevad reaalsetel andmetel ja mis näitavad, kui oluline on arvestada sildade haldamisel määramatusega. Näited on koostatud Eesti riigiteedel enam levinud sildadelt kogutud andmete alusel, mida Markovi ahelate Monte Carlo algoritmide simulatsiooni tulemusena väljendatakse tõenäosuslike seisukorra väärtustena ja mille uuendamisel kasutatakse Bayesi meetodit. Tulemused kinnitavad, et seisukorra prognoosimise ja hindamisega kaasnevad kõrvalekalded mõjutavad elemendi seisunditaseme määramatust. Viimane omakorda mõjutab planeeritud ülevaatuse ja parendustegevuse aega ja seeläbi üldisi kulusid ning otsuste usaldusväärsust.

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