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Assessment of Existing Bridges in Norway

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Abstract

The Norwegian Public Roads Administration maintains the road network in Norway. The administration has its own guidelines for the inspection and capacity assessment of bridges, of which maintenance decisions are based on. With a large portfolio of ageing and deteriorating bridges, an efficient and systematic way for deciding on optimal methods and timing for the maintenance of bridges is very valuable for bridge managers.

The objective of this thesis is to explore and illustrate the potential that a decision making framework concerning assessments of existing bridges in Norway would bring. In the study, an emphasis was put on reinforced concrete structures subject to deterioration due to carbonation induced corrosion and chloride induced corrosion. Common condition assessment and repair and rehabilitation methods were reviewed. An example was established to illustrate the process of determining the optimal inspection and repair strategy using Bayesian decision analysis. Current inspection practices at the administration were identified and an interview was conducted with an employee to get further insight into their methods. Service life modelling was introduced as a tool that can be used to predict damage development and to plan maintenance measures. Recent inspection and capacity assessments of an existing bridge performed by the administration were presented. Finally, an example illustrating the application of the service life modelling approach for an existing bridge was established.

The study shows that the Bayesian decision method is effective to find an optimal solution in a consistent way. The inspection practices currently used by the Norwegian Public Roads Administration can be improved by implementing probabilistic methods. Risk and vulnerability analyses should be a standardised routine linked to the inspection of every bridge in the system. Predictive deterioration models have the potential to guide decision making in bridge management. The models can be updated by incorporating inspection data, resulting in better accuracy. Same goes for the capacity assessment method used at the Norwegian Public Roads Administration, which can be improved by using probabilistic methods with updating of parameters through inspection results. The probabilistic models can be adapted to include deterioration.

Acknowledgements

Firstly, I would like to thank my supervisors professor Jochen Köhler and PhD candidate Frida Liljefors, for giving me the opportunity to conduct this study, and for a fruitful cooperation and guidance throughout my work. I want to thank everyone at the Norwegian Public Roads Administration who devoted their time in order to help me. Special thanks goes to Tor Anders Hagstrøm, who provided me with great insight on bridge assessment procedures at the road administration. Finally, I would like to thank my family and friends for all their support and encouragement throughout my studies.

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Executive Summary

The main results from this thesis are presented in this executive summary.

Introduction

Infrastructure owners have to manage and maintain a large portfolio of bridges. They are faced with the challenge of how to efficiently manage a stock of ageing and deteriorating bridges under today's heavy traffic conditions, while having a limited budget. This challenge addresses the need for a systematic way for deciding on optimal methods and timing for the repair and maintenance of bridges. The Norwegian Public Roads Administration (NPRA) maintains the road network in Norway. The administration has a large stock of concrete bridges and for this reason, the study focuses on deteriorating concrete bridges. An emphasis is put on reinforced concrete structures subject to deterioration due to carbonation induced corrosion and chloride induced corrosion. Bridge inspections are a crucial part of maintaining and ensuring the safety of bridges. Analysing the present condition and future development of deterioration leads to the determination of repair and maintenance strategies for the bridge. The administration has its own guidelines for the inspection and capacity assessment of bridges, of which maintenance decisions are based on.

The goal of this thesis is to explore and illustrate the potential that a decision making framework concerning assessments of existing bridges in Norway would bring. The findings should be of interest for organisations responsible for the management of bridges, such as NPRA, counties, municipalities, and other bridge owners, and for organisations such as consultants, that perform the load-carrying capacity assessments of the bridges.

Findings

To support decisions, probabilistic methods can be used. Bayesian decision theory enables rational and coherent basis for decision making regarding bridge inspection, repair, and maintenance, when uncertainties are involved in the process. With this approach, the decision maker can determine which information in necessary to make the optimal decision. Using this method can lead to the optimisation of decision making.

The inspection practices currently used by NPRA can be improved by implementing probabilistic methods. These methods show a good potential for guiding decision making in bridge inspections and assessments and reduces inconsistency. Fixed inspection intervals fail to consider the need for inspections for bridges in varying conditions, leading to inspection resources not being effectively utilised. By having verified risk and vulnerability analysis as a routine linked to the inspection of every bridge in the system, inspections for bridges in good conditions are delayed, making resource management more effective. For bridges that are in poor condition, this could mean that inspection intervals become shorter and damages identified sooner, which would potentially avoid costly measures of repair or maintenance.

Predictive deterioration models are a good tool to guide decision making in bridge management. The models can be used to forecast the need for maintenance, and their benefits include that the most appropriate repair or maintenance strategy is performed at the right time during the service life of the bridge. Non-visible damages can potentially be detected earlier, making it possible to reduce the need for extensive and expensive repair. Parameters used in the service life models can be obtained through routine inspections. Collecting inspection data systematically would be valuable, as data collected throughout the years can be further used to improve the quality of deterioration models.

Bayesian updating can be used for the improvement of deterioration models which are repeatedly updated as the inspection data is incorporated. Better understanding of the condition of the bridge would be obtained, maintenance and repair scheduling improved, and resources would be managed more efficiently. The chosen input data can have a large influence on the results of decision making. The selection of reliable values for the input parameters is thus important.

The capacity assessment method used by NPRA which is based on the Eurocode partial safety factor method can lead to lack of accuracy and conservative results. With a large stock of bridges, resources are wasted if the same conservative method is followed for all bridges. Instead, more detailed probabilistic methods may be used. They result in more accurate results and saving of resources. This would be a more time consuming and expensive approach, however, with probabilistic methods, new information can be accounted for and limit states adapted to take deterioration into account. There is often the situation where bridges need to be reassessed. It can thus be of importance that the method used for reassessment of bridges gives a more accurate result. Inspection results can be incorporated into capacity assessments for a more accurate result of the load-carrying capacity of the bridge to be obtained. Consequently, the management of bridges would become more efficient. As for service life prediction models, the results from a probabilistic assessment depends strongly on the assumptions made about the uncertainties associated with parameters. A 'complete' probabilistic approach would be a structural reliability analysis of the load-carrying capacity, adapted in such a way that the reduction of reinforcement would be considered, for example from service life modelling.

Recommendations for Further Work

The results from this thesis set out the potentials that a decision making framework would bring for the assessments of existing bridges. However, further research is required before such a framework can be set. The recommendations provided from this work are as follows:

- Adopt Bayesian decision analysis approach to current practises for the optimisation of inspection, repair, and maintenance of existing bridges. The simple concepts of the approach should be studied;

- Have verified risk and vulnerability analysis as a routine procedure linked to the inspections of bridges at the Norwegian Public Roads Administration;

- Use predictive service life models to guide decision making in bridge management. The models would be used as a basis for the determination of repair and maintenance strategies. For the models to give realistic results, the selection of input parameters needs to be evaluated. In addition, further work needs to be done on modelling the service life of deteriorating structures for multiple deterioration mechanisms;

- Explore the option of using a combination of semi-probabilistic methods and reliability based methods based on a probabilistic approach, in the assessment of existing structures. The probabilistic methods can be used as an additional option. The models can be adapted to include deterioration. The use of more sophisticated models can be helpful and provide a more reliable basis for decision making;

- Incorporating inspection results into capacity assessment models and prediction service life models for the improvement of their accuracy. Reliable methods for obtaining inspection data are required. What kind of inspection data can be used for the updating of models needs to be investigated.

1 Introduction

1.1 Background

1.1.1 Assessing Existing Structures

The bridge population around the world is ageing. Bridges are subject to harsh environments which leads to their deterioration. At the same time, number of vehicles on the road, loads, and legal load limits have been steadily increasing. Ageing bridges subject to increasing legal load limits means that they often fail to satisfy structural requirements as specified for new bridges. The population of ageing bridges leads to decreased safety and increased functioning costs, which can have environmental, social, and economic consequences (Stewart 2001).

A fundamental cause for bridge deficiencies is insufficient maintenance. Existing damages on bridges, along with increasing traffic loads and harsh environmental conditions, lead to rapid deterioration of bridge elements, which may require immediate maintenance actions (Chassiakos et al. 2005).

Infrastructure owners have to manage and maintain a large portfolio of bridges. They are faced with the challenge of how to efficiently manage a stock of ageing and deteriorating bridges under today's heavy traffic conditions, while having a limited budget (Flaig et al. 2008). For bridge design, it is acceptable to be conservative since the additional material costs are marginal. However, a conservative bridge assessment results in unnecessary and costly bridge repair or strengthening, traffic disruption, and so forth (Stewart 2001). The large number of bridges in the road network and high maintenance costs justify the development of sophisticated approaches to bridge management (Chassiakos et al. 2005).

1.1.2 Decision Making

The development and management of infrastructure is fundamental for the success of society. Decision processes in this regard are used to improve the quality of life of the individuals of society. If all aspects of a decision problem are known with certainty, the identification of the optimal solution would be straightforward. However, the decision problems in engineering are usually subject to significant uncertainty, making it impossible to assess with certainty the consequences resulting from the decisions made (JCSS 2008). In decision problems such as assessment of existing structures, evaluating the most applicable maintenance strategy for a large portfolio of bridges is a complex procedure, as there are several parameters that determine the most economical solution (Chassiakos et al. 2005). Therefore, a systematic analysis of decision making problems can be very beneficial (JCSS 2001).

1.1.3 Bridge Management

Brutus is the bridge management system currently used by Norwegian Public Roads Administration (NPRA) to collect condition data about bridges maintained by the administration. According to the system, NPRA is maintenance responsible for 6,124 bridges that are registered as 'in traffic'. Of those bridges, 4,355 are labelled as 'concrete' bridges and 772 as 'prestressed concrete' bridges, making it a total of 5,127. Table 1 shows the age distribution of the labelled 'concrete' bridges:

Table 1: Year of construction of concrete bridges registered in Brutus (NPRA n.d. *a*).

Year	**Number of bridges**
-1959	356
1960-1969	532
1970-1979	656
1980-1989	698
1990-1999	847
2000-2009	557
2010-2019	670
2020-	21
Unknown	18
Total	4,355

With so many ageing concrete bridges in the road network that NPRA manages, this thesis focuses on deteriorating concrete bridges. In addition, the number of bridges that are managed by counties and municipalities in Norway are 12,356 and 2,317, respectively.

1.1.4 Maintenance Needs and Public Perception of Bridge Safety in Norway

The safety of structures is perceived by society as a basic requirement (JCSS 2001). Concern can arise if a structure feels or looks unsafe. The perception of safety is an area where structural engineering, bridge management, human psychology, and public relations meet (Oliver 2021).

In April 2022, a news article was published by NRK, the government owned radio and television public broadcasting company, addressing concerns for structural safety of bridges in Norway (Siem 2022). According to the article, 1,016 bridges in Norway have damages that are classified as having 'considerable consequence' for the load-carrying capacity of the bridges and 51 bridges 'great consequence'. In Vestland county, there are over 2,000 bridges, whereas on many of them the condition is critical in terms of ensuring access and load-carrying capacity. In the article, residents in Vestland express that they are worried about the condition of the bridges, and that there is a need for repair and maintenance throughout the country. The condition of bridges is even a concern for politicians. The mayor of Bremanger municipality in Vestland, Anne Kristin Førde, agrees with the residents and emphasises that the local community is completely dependent on some of the bridges. The department director for infrastructure and roads in Vestland, Dina Lefdal, mentions that there is large backlog of maintenance of bridges, something that worries them in their daily lives. With no maintenance measures, the allowed load on the bridges must be reduced and the bridges possibly closed. The leader of the Standing Committee on Transport and Communications in Norway, Erling Sande, says that the lack of maintenance is a serious concern. He claims that the maintenance of bridges in Norway should be prioritised and that some of the largest road and railway construction projects in Norway should be scaled down in order to have funds for the maintenance of county roads, bridges, tunnels, and landslide mitigation and protection (Siem 2022).

Another recently published article also discusses the seriousness of bridges decaying in Vestland (Ramstad 2022). Bømla bridge (NO: Bømlabrua), is mentioned as one of the bridges in Vestland that need immediate, extensive, and expensive maintenance. A special inspection of Bømla bridge revealed that the large steel support cables are in the process of being classified with 'critical damage' due to moisture damages that would effect the load-carrying capacity, if they are allowed to develop further. Bømla bridge is shown in fig. 1.

Figure 1: Bømla bridge in Vestland county in Norway. The bridge is in need of extensive and expensive maintenance (Ramstad 2022).

The bridge, which was built in 2001, lacks dehumidification system for the steel cables. Now the consequences are increased maintenance costs and possible closure of the bridge if measures are not implemented. Funds have now been approved for necessary and instant maintenance for the steel cables. The estimated costs for the immediate maintenance measures at Bømla bridge are 85 MNOK.

In the article, the county mayor, Jon Askeland, comments on the urgency of this maintenance. He emphasises that many of the bridges in the county need maintenance. The county municipality has registered 778 bridges on the county road network with major or very large damage, 87 bridges with corresponding damage affecting load-carrying capacity, and 444 bridges with a large or very large need for maintenance to prevent damage that affects the load-carrying capacity of the bridge (Ramstad 2022).

Even more recently, on May 31st 2022, a bridge in North Troms (in Northern Norway) collapsed due to flooding, causing the bridge to be closed to all traffic. Bridge experts at NPRA concluded that the damage is so severe that the bridge has to be replaced. It took one week to construct a temporary bridge over the damaged bridge, so that traffic could go through (Pedersen et al. 2022).

The bridge closing had large consequences, both for the local population and the transport industry. Until the temporary bridge was set up, all traffic had to detour through Finland, which was extra 700 km of driving. The detour through the neighbouring country lead to extra long driving distances, extra rest time for the drivers, and thus increased costs. Necessary medicines was transported by boat and a speedboat was used to transport students to school (Malmo et al. 2022). Even the Norwegian Armed Forces arrived at the scene to see if a military transfer would be possible if necessary, see fig. 2 (Pedersen et al. 2022).

Figure 2: The Norwegian Armed Forces arrived at the scene to evaluate if military transfer would be possible if necessary (Pedersen et al. 2022).

After this collapse, the government has been criticised for not securing the road network in the districts. The situation shows the vulnerability of the infrastructure in Northern Norway, says the regional manager of the Norwegian Truck Owners Association, Odd-Hugo Pedersen (Malmo et al. 2022). The deputy leader of the Green Party, Arild Hermstad, says that the bridge collapse is a clear example that the road network in the district is not a priority. He believes that there is still too little priority given to resources to improve the road network in the district, and criticises the current government for prioritising new construction projects (Malmo et al. 2022).

1.2 Scope and Objectives

The scope of this thesis is to explore and illustrate the potential of a framework for decision making, to enable a rational and consistent basis for decisions concerning assessment of existing bridges in Norway. The aim is to list the potential benefits of such a framework.

One of the objectives is to identify current inspection practices at NPRA to provide an insight into their methods and to demonstrate how their practise can be improved. The study explores how probabilistic methods can be used for bridge analyses, and how inspection results can be incorporated into deterioration models with Bayesian updating. With this, the aim is to more accurately predict the load-carrying capacity of structures and to schedule repair and maintenance activities, in an optimal way.

The subjects of discussion are deterioration mechanisms and how they affect the structural behaviour of concrete structures, probabilistic models for selected deterioration mechanisms, Bayesian decision theory, and inspection processes. A recent capacity assessment made by NPRA is presented and an example showing how a fully probabilistic service life model can be used for decision making is demonstrated.

The findings should be of interest for organisations responsible for the management of bridges, such as NPRA, counties, and bridge owners, and for other organisations such as consultants that perform the load-carrying capacity assessments of the bridges.

1.3 Outline of the Thesis

The thesis consists of 9 chapters. The structure of each chapter after the introduction is as follows:

Chapter 2 describes deterioration of concrete structures. Concrete deterioration caused by carbonation induced corrosion and chloride induced corrosion are discussed in detail. Condition assessment methods along with repair and rehabilitation methods are reviewed;

Chapter 3 illustrates how Bayesian decision theory can be used as a support for decision making regarding bridge inspections, repair, and maintenance, when uncertainties are involved in the process;

Chapter 4 provides an insight into inspection practises developed by NPRA;

Chapter 5 further deals with inspection practises at NPRA by presenting the main conclusions from an interview conducted with an employee in the Bridge and Ferry Qauy department at the administration;

Chapter 6 gives an introduction to durability design and how service life modelling can be used to assess the remaining service life of existing structures.

Probabilistic models developed by *fib* Bulletin 34 (2006) for the depassivation of reinforcement caused by carbonation and chloride attack are presented;

Chapter 7 presents the findings from an inspection report and capacity assessment made by NPRA for a bridge located in Northern Norway. An example is demonstrated to show how a service life model can be used to assess a bridge while incorporating inspection results for the update of the model;

Chapter 8 gives a general discussion on the main findings of the thesis;

Chapter 9 presents the conclusions of the research described in the thesis and provides recommendations for further work.

2 Deterioration of Concrete Structures

2.1 General

Reinforced concrete (RC) is a widely used construction material for structures such as bridges, buildings, and tunnels. Generally, RC is a very durable material that can withstand a large range of severe environments (Hunkeler 2005).

The ability of concrete structures to fulfil their intended functions, such as serviceability, durability, and structural safety, can be compromised due to deterioration. Harsh environments for concrete structures, for example caused by chloride ingress or carbonation, are a major cause of performance degradation in reinforcement (Chen & Alani 2013). Bertolini et al. (2013) classifies the degradation processes as physical, mechanical, chemical, biological, and structural. In practice, these processes may occur simultaneously. The processes are shown in fig. 3. As can be seen, the processes of concrete and reinforcement corrosion are closely connected. Degradation of concrete provokes destruction of the concrete cover and/or causes micro-cracking that reduces the protective functionality of the concrete. Reinforcement corrosion can also produce cracking or delamination of the concrete (Bertolini et al. 2013).

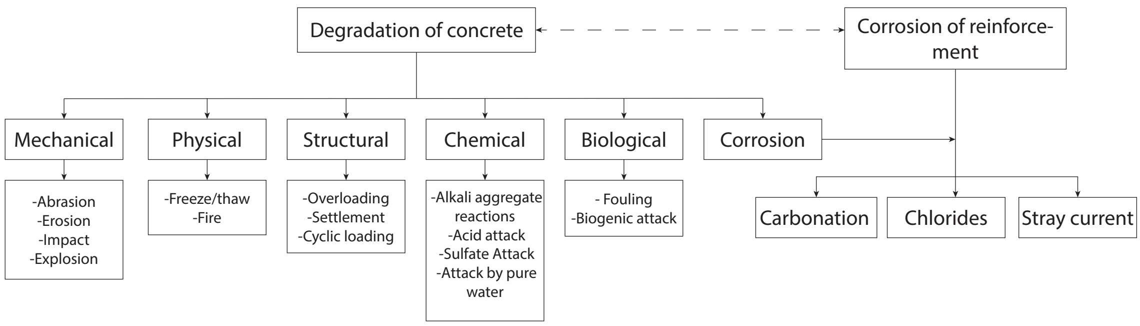

Figure 3: Causes of deterioration of RC structures. Based on Bertolini et al. (2013).

2.2 Corrosion of Steel Reinforcement in Concrete

2.2.1 Introduction

Corrosion of reinforcing steel has been identified as being the predominant deterioration mechanism for RC structures. Reinforcement corrosion can seriously affect structural safety (Markeset & Myrdal 2009). Particularly, long-term durability of RC structures are threatened by carbonation induced and chloride induced corrosion (Zhou et al. 2015). The mechanisms of carbonation induced and chloride induced corrosion will be further discussed in section 2.2.3.

2.2.2 General Aspects

Concrete is alkaline in nature with a pore solution pH of 12-13 (Zhou et al. 2015). This environment causes a protective passive film to form on the surface of the steel reinforcement. This passive film is mostly composed of hydrated iron oxides with varying ratios between Fe^{2+} and Fe^{3+} (Bertolini et al. 2013). The protective action of the passive film is immune to mechanical damage of the steel surface, even if sufficient moisture and oxygen are available. As long as the passive film remains intact the corrosion reaction is at a very low, insignificant rate (Markeset & Myrdal 2009). However, the layer can be destroyed by carbonation of concrete or by the presence of chloride ions, and the reinforcing steel is then depassivated (Bertolini et al. 2013).

In order for initiation of corrosion, the passive layer must be broken down. The protective layer is destroyed if the pore water in contact with the steel drops to a pH of about 9 (usually as a result of carbonation) and/or the pore water in contact with the steel contains dissolved chloride ions above a certain threshold level (Markeset & Myrdal 2009). Once the layer is destroyed, corrosion will only occur if water and oxygen are present on the surface of the reinforcement (Bertolini et al. 2013).

Initiation and Propagation of Corrosion

The service life of a RC structure can be divided into two phases, the initiation phase and the propagation phase, as illustrated in fig. 4. During the initiation phase, aggressive substances, such as carbon dioxide and chlorides, that can depassivate the steel, penetrate from the surface into the bulk of the concrete. The duration of the initiation phase depends on factors such as the cover depth, penetration rate of the aggressive substances, and the concentration necessary to depassivate the steel. The larger the cover, the longer it takes for the substances to reach the reinforcement, hence the initiation period is longer. The rate of ingress depends on the quality of the concrete cover (porosity and permeability) and on the micro-climatic conditions (wetting, drying) on the concrete surface. Measures can be used to prolong the initiation phase. The initiation time can also be affected by any form of polarisation of the reinforcement (Bertolini et al. 2013).

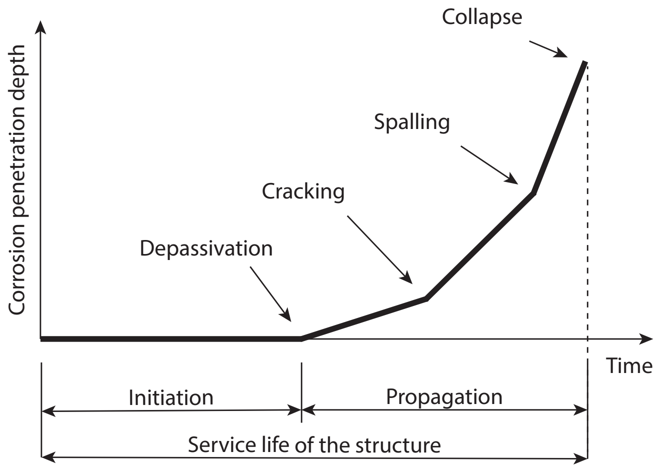

Figure 4: Initiation and propagation phases for reinforcement corrosion in a concrete structure (Bertolini et al. 2013).

The propagation phase begins when the steel is depassivated (i.e. corrosion initiates). During the propagation phase, the reinforcement is corroding, which can lead to the deterioration of concrete as well (Bertolini et al. 2013). Expansive corrosion products provoke cracks along the reinforcement, which can cause spalling of the concrete cover. Reduction of the reinforcement cross section due to corrosion can decrease the load-carrying capacity of the structure (Markeset & Myrdal 2009). The propagation phase ends when a limit state is reached beyond which consequences of corrosion cannot further be tolerated (Bertolini et al. 2013).

The Corrosion Process

Iron and plain carbon steel (iron containing a small percentage of carbon) are thermodynamically unstable materials. Nature will try to bring these materials back to their thermodynamically stable state, iron-oxides, i.e. rust. Under neutral and basic conditions, as in concrete, the steel must have physical contact with water and oxygen in order to corrode. The water and oxygen act as chemical 'driving forces' (Markeset & Myrdal 2009). The corrosion process can be summarised with the following reaction:

$$\text{iron} + \text{oxygen} + \text{water} \longrightarrow \text{corrosion products} \tag{1}$$

The corrosion process involves two seperate, but coupled, half-cell reactions that take place simultaneously at two different sites on the steel surface. These reactions are called anodic reaction (oxidation of iron, eq. (2)) and cathodic reaction (reduction of oxygen, eq. (3)) and the areas on which they occur in the steel are called 'anodic' and 'cathodic' areas, respectively. An electric current must flow in a closed loop between the two sites for the reactions to proceed (Bentur et al. 2005).

$$\mathrm{Fe} \longrightarrow \mathrm{Fe}^{2+} + 2\mathrm{e}^{-} \tag{2}$$

$$\frac{1}{2}O_2 + H_2O + 2e^- \longrightarrow 2OH^- \tag{3}$$

In the anodic reaction the iron atoms are ionized to ferrous ions which dissolve in the water solution around the steel. The electrons are deposited on the steel surface and raise its electrical potential. The electrons then flow through or along the steel to a lower potential (cathodic) site. In the cathodic reaction, the electrons combine with dissolved oxygen molecules and water to form hydroxide (OH^-) ions. In order for the corrosion process to continue, the number of electrons accepted at the cathodic site needs to be equal to the number of electrons donated at the anodic site (Bentur et al. 2005). A schematic illustration of the corrosion process is shown in fig. 5.

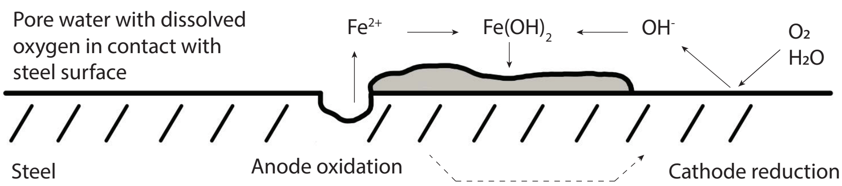

Figure 5: A schematic illustration of a steel corrosion cell in concrete. Based on Markeset & Myrdal (2009).

For rust to form, several more stages must occur, which can be expressed in several ways. Broomfield (2007) describes the process where ferrous hydroxide (eq. (4)) becomes ferric hydroxide (eq. (5)) and then hydrated ferric oxide or rust (eq. (6)):

$$Fe^{2+} + 2OH^- \longrightarrow Fe(OH)_2 \tag{4}$$

$$4Fe(OH)_2 + O_2 + 2H_2O \longrightarrow 4Fe(OH)_3 \tag{5}$$

$$2Fe(OH)_3 \longrightarrow Fe_2O_3H_2O + 2H_2O \tag{6}$$

The rust is a chemical by-product of the corrosion process, and it often accumulates at places other than where the actual corrosion of the steel occurs. In general, the volume of rust produced in a corrosion reaction is at least twice the volume of the steel that is dissolved. Therefore, rust formation involves a substantial volume increase (Bentur et al. 2005). Figure 6 shows relative volume of iron and its oxides that are formed as result of reinforcement corrosion.

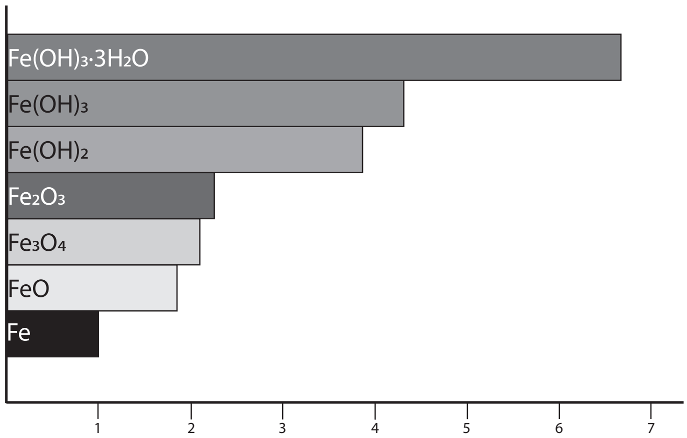

Figure 6: Relative volume of iron and its oxides. Based on Broomfield (2007).

Rate of Corrosion

In analysing the practical aspects of steel corrosion in concrete, an aspects that needs to be addressed is that if corrosion occurs, at what rate will it proceed (Bentur et al. 2005). The corrosion rate determines the time it takes to reach an undesirable event such as loss of cross section of the reinforcement, cracking of the concrete cover, spalling and delamination of the concrete cover, and, eventually, collapse (Bertolini et al. 2013). In practical terms, one wants to understand how much of the steel cross section will be lost per year, and what rate of accumulation of corrosion products may be expected. The actual rate of corrosion may even be so slow that the corrosion processes are of little practical significance (Bentur et al. 2005).

The corrosion rate is usually expressed as the penetration rate and is measured in μm/year. In laboratory tests it is often expressed in electrochemical units (mA/m^2 or μA/cm^2). For steel, 1 μA/m^2 = 0.1 μA/cm^2 corresponds to a mass loss of approximately 9 g/m^{2}year and a penetration rate of about 1.17 μm/year. Bertolini et al. (2013) divided the corrosion rate as shown in table 2.

Table 2: Corrosion rate and level of corrosion. Based on (Bertolini et al. 2013).

Corrosion rate [μm/year]	**Corrosion level**
<2	negligible
2-5	low
5-10	moderate
10-50	intermediate
50-100	high
>100	very high

Active Corrosion

Once the passive layer is destroyed, the corrosion rate usually increases by several orders of magnitude, and the corrosion process will take place (Markeset & Myrdal

2009). Two main types of active corrosion are seen in concrete: general corrosion and pitting corrosion.

General Corrosion

Uniform corrosion attack over large areas on the surface of the reinforcement. This type of corrosion is a typical characteristic of corrosion initiated by pH drop (carbonation) (Markeset & Myrdal 2009). In exceptional cases where high levels of chlorides are present (or the pH decreases), the passive film can be destroyed over large areas of the reinforcement and the corrosion will be of a general nature (Markeset & Myrdal 2009, Bertolini et al. 2013).

Pitting Corrosion

Involves a local anodic dissolution of iron, surrounded by large areas of passive steel bar acting as the cathode. Pitting corrosion is a typical characteristic of chloride induced corrosion (Markeset & Myrdal 2009). Chloride ions cause localised breakdown of the passive film that initially was formed on the steel reinforcement as a result of the alkaline nature of the pore solution in concrete, so that a subsequent localised corrosion attack takes place. Areas no longer protected by the passive film act as anodes with respect to the surrounding still passive areas where the cathodic reaction takes place. The morphology of the attack is that typical of pitting (Bertolini et al. 2013). Chloride induced pitting corrosion is schematically illustrated in fig. 7 (Markeset & Myrdal 2009).

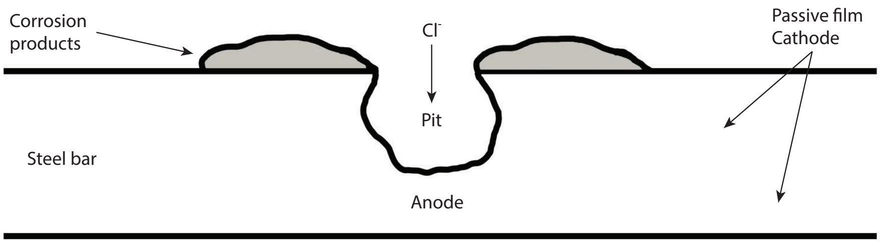

Figure 7: A schematic illustration of chloride induced pit formation on steel in concrete. Based on Markeset & Myrdal (2009).

Once corrosion has initiated, a very aggressive environment will be produced inside the pits. Current flowing from anodic areas to surrounding cathodic areas increases the chloride content and lowers the alkalinity. The current however strengthens the protective film on the passive surface since it tends to eliminate the chlorides, while the cathodic reaction produces alkalinity. As a result, the anodic behaviour of active zones and the cathodic behaviour of passive zones are stabilised. Then, corrosion is accelerated, and can reach very high penetration rates, up to 1 mm/year. With such a high penetration rate, the cross sectional area can be reduced quickly, without having signs of cracking or spalling, making it difficult to identify during visual inspection of the structure.

2.2.3 Causes and Mechanisms of Steel Corrosion in Reinforced Concrete

Carbonation and chloride attacks are the two main causes of corrosion of reinforcement steel in concrete. These two mechanisms do not attack the integrity of the concrete but instead, aggressive chemical species pass through the pores of the concrete and cause damage to the steel (Broomfield 2007). This section will discuss how these mechanisms will lead to corrosion.

Carbonation Induced Corrosion

General

Carbonation is a process where carbon dioxide (CO_2) in the air forms an acid aqueous solution and reacts with hydrated cement paste, neutralising the alkalinity of concrete (Bertolini et al. 2013). The CO_2 gas dissolves in water and forms a carbonic acid (H_2CO_3). The acid neutralises the alkalies in the pore water of the concrete, mainly forming calcium carbonate ($CaCO_3$) (Broomfield 2007). The reaction can be schematically written as:

$$CO_2 + H_2O \longrightarrow H_2CO_3 \tag{7}$$

$$H_2CO_3 + Ca(OH)_2 \longrightarrow CaCO_3 + 2H_2O \tag{8}$$

There is a lot more calcium hydroxide ($Ca(OH)_2$) in the concrete pores that can dissolve in the pore water, which helps maintaining the natural pH level of the concrete when the carbonation reaction starts. However, as CO_2 continues to react with $Ca(OH)_2$ and other hydroxides in the pore solution, eventually, all the $Ca(OH)_2$ reacts, precipitating the calcium carbonate. This causes the pH to drop layer by layer in the concrete until it reaches the reinforcement steel where it starts to corrode (Broomfield 2007). An illustration of a carbonation reaction penetrating into the concrete is shown in fig. 8.

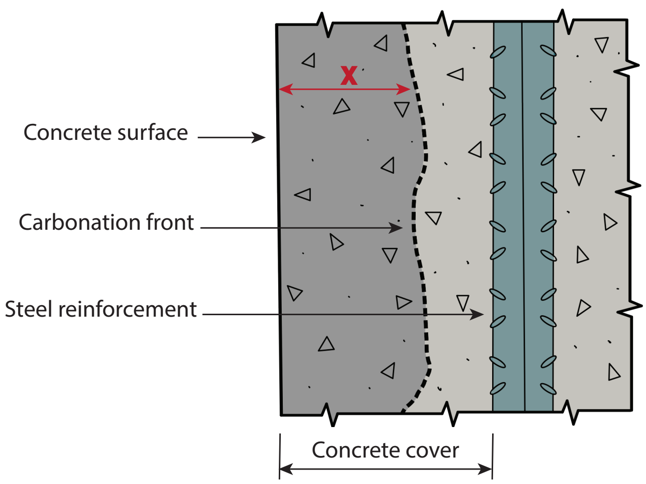

Figure 8: Carbonation front penetrating into the concrete. Based on Segui et al. (2017).

Rate of Carbonation

The carbonation reaction starts at the external surface and penetrates into the concrete, roughly following Fick's law of diffusion, where the rate of movement is considered to be proportional to the distance from the surface (Broomfield 2007):

$$\frac{dx}{dt} = \frac{D_0}{x} \tag{9}$$

where x is the distance, t is time, and D_0 is the diffusion constant. The rate of carbonation decreases with time, as CO_2 has to diffuse through the pores of the already carbonated outer layer (Bertolini et al. 2013). This penetration can be described by:

$$d = K \cdot t^{1/\mathrm{n}} \tag{10}$$

where d is the depth of carbonation [mm] and t is time [years]. K is the carbonation coefficient [mm/year$^{1/2}$], which can be described as a measure of the rate of penetration of carbonation for given concrete and environmental conditions. Often the exponent n equals 2, so the penetration can be considered to be:

$$d = K \cdot \sqrt{t} \tag{11}$$

The rate of carbonation depends on factors related to the concrete and the environment (Zhou et al. 2015, Bertolini et al. 2013). Those include cover thickness (good cover depth is essential to resist carbonation), carbonation resistivity, effective CO_2 diffusion coefficient, binding capacity for CO_2, curing condition, age, cement type, cement composition, calcium oxide (CaO) content in cement, surface concentration of CO_2, time of wetness, ambient temperature, and relative humidity. Environmental conditions, such as sheltered versus exposed and underground versus atmospheric, also have an important impact on concrete carbonation process (Zhou et al. 2015).

Chloride Induced Corrosion

Considering marine exposure conditions and the extensive use of de-icing salts, chloride induced corrosion is one of the most common causes of degradation of RC structures (Angst et al. 2009). In many countries it is regarded as the most important degradation mechanism for RC infrastructure (Bertolini et al. 2013).

The depassivation mechanism for chloride induced corrosion is different from the mechanism for carbonation induced corrosion. There is no overall drop in the pH level of concrete. Instead, the chlorides act as catalysts to corrosion as they help to break down the passive layer of oxide on the steel and allow the corrosion process to proceed quickly. A few chloride ions in the pore water will not break down the passive layer, but there is a certain chloride threshold value necessary to sustain local passive film breakdown (Broomfield 2007).

Critical Chloride Content

Reinforcement corrosion in non-carbonated, alkaline concrete can only start once the chloride content at the surface of the reinforcement has reached a certain threshold value. This value is often referred to as 'critical chloride content' or 'chloride threshold value' (C_{crit}). Two definitions have been used for this value: (1) the chloride content required for depassivation of the steel, or (2) the chloride content associated with visible/acceptable deterioration of the RC structure. Definition (1) only considers the initiation stage where as for definition (2) the propagation phase has also started (Angst et al. 2009).

C_{crit} according to definition (1) is considered as the chloride content measured at the depth of the reinforcement that causes depassivation of the reinforcement (Bertolini et al. 2013). Similar to carbonation depth, the chloride threshold value depends on a number of material and environmental related factors. They include the cover depth, chloride resistivity, and chloride binding capacity of concrete, water-binder ratio of the concrete mixture, curing conditions, age of concrete, cement type, cement composition, surface chloride concentration, ambient temperature, and relative humidity. The geographic environment of the structure, such as inland zone, coastal region, or marine, also has a significant impact (Zhou et al. 2015). In structures exposed to the atmosphere, where oxygen can easily reach the reinforcement, relatively low levels of chlorides are necessary to initiate corrosion. In structures immersed in seawater or zones where the concrete is water saturated, much higher levels of chlorides are necessary to initiate corrosion because the oxygen supply is hindered, thus the potential of the reinforcement is rather low (Bertolini et al. 2013).

Chloride Penetration

Chlorides from the environment produce a concentration profile in the concrete, which is characterised by high chloride content near the external surface and a lower chloride content in the deeper region of the concrete. Experience on marine structures and road structures exposed to de-icing salts has shown that chloride profiles can be simplified and estimated by Fick's second law of diffusion (Bertolini et al. 2013):

$$\frac{\partial C_{\mathrm{Cl}}}{\partial t} = D_{\mathrm{app}} \frac{\partial^2 C_{\mathrm{Cl}}}{\partial x_{\mathrm{Cl}}^2} \tag{12}$$

where C_{Cl} is the total chloride content [% by mass of cement of concrete], at time t [s], distance x_{Cl} [m] from the surface of the concrete, and D_{app} [m^2/s] is the apparent diffusion coefficient for chloride. This equation is solved by applying boundary conditions:

$$C_{\mathrm{Cl}}(x_{\mathrm{Cl}} = 0, t > 0) = C_{\mathrm{s,Cl}} \tag{13}$$

$$C_{\mathrm{Cl}}(x > 0, t = 0) = 0 \tag{14}$$

That is, the concentration of the diffusing ion, measured on the surface of the concrete, is constant in time and is equal to the surface concentration of chloride, $C_{s,Cl}$ [% by mass of cement or concrete], that the coefficient of diffusion D_{app} does not vary in time, that the concrete is homogeneous, so that D_{app} does not vary through the thickness of the concrete, and that it does not initially contain chlorides (Bertolini et al. 2013). Thus, the obtained solution is:

$$C(x,t) = C_{\mathrm{s,Cl}}\left(1 - \mathrm{erf}\left(\frac{x}{2\sqrt{D_{\mathrm{app}} \cdot t)}}\right)\right) \tag{15}$$

where

$$\mathrm{erf(z)} = \frac{2}{\sqrt{\pi}} \int_0^z e^{\mathrm{-t}^2} dt \tag{16}$$

is the error function. The long-term performance of structures exposed to chloride environments can be predicted using eq. (15), for example in the design stage of a structure or during an assessment of existing structure. With estimated apparent diffusion coefficient, D_{app}, and surface chloride content, $C_{\mathrm{s,Cl}}$, and with the assumption that they are constant in time, it is possible to estimate the time t at which a particular chloride content will be reached. This approach is simple but it should be observed that it strongly depends on the reliability of the parameters distributed, including the chloride threshold value. In real structures, where processes other than diffusion take place, D_{app} and $C_{\mathrm{s,Cl}}$ cannot, generally, be assumed to be constant (Bertolini et al. 2013).

2.3 Consequences of Corrosion

Reinforcement corrosion, induced by carbonation and/or chlorides, causes degradation of RC structures. If reinforcement corrosion is initiated, the consequences that result can be related to several aspects regarding the structure's aesthetic appearance, serviceability, safety, and structural performance (Bertolini et al. 2013).

Corrosion is often indicated by rust spots that appear on the external surface on the concrete, or by cracking of the concrete cover, which is due to expansion caused by corrosion products, as mentioned in section 2.2.2. The corrosion products can produce tensile stresses that generate cracks in the concrete cover, spalling in localised areas, or complete delamination, examples of which can be seen in fig. 9 (Bertolini et al. 2013). The cracks can reduce the load-carrying capacity, shorten the service life, and increase the rate of ingress of aggressive substances (Zhou et al. 2015).

Figure 9: Examples of consequences of corrosion of steel in concrete (Bertolini et al. 2013).

The most direct damage resulting from reinforcement corrosion in a conventionally RC structure is the reduction in steel diameter and cross sectional area, where the consequence is that the magnitude of stresses carried by the remaining steel naturally increases (Bentur et al. 2005). These reductions could lead to a premature fracture of the reinforcement bar before yielding is observed (Zhou et al. 2015).

In addition, corrosion significantly influences the interface bond behaviour between concrete and rebar. Zhou et al. (2015) mentiones that it has been shown that the bond strength first increases and then decreases with increasing level of corrosion.

In the case of pitting corrosion, the reinforcement cross section and thus its load-carrying capacity and fatigue strength can be significantly reduced long before any sign of corrosion becomes detectable at the concrete surface. Consequences of pitting corrosion can be very serious in high-strength, pre-stressing steel, where hydrogen embrittlement can occur (Bertolini et al. 2013). An example of a steel bar suffering localised corrosion attack is shown in fig. 10.

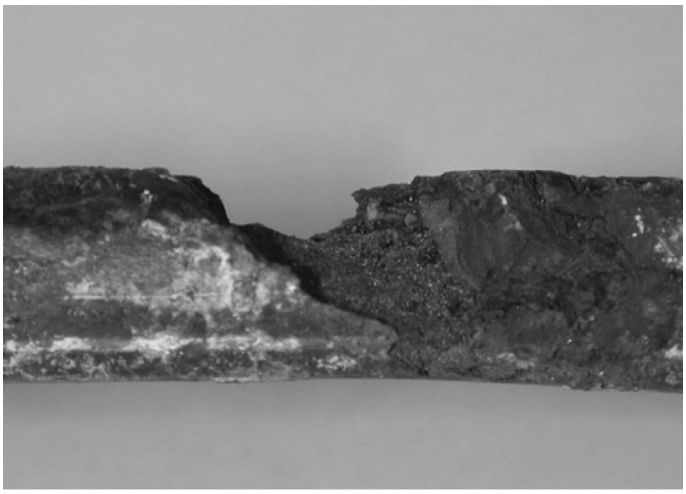

Figure 10: An example of a steel bar suffering localised corrosion attack (Bertolini et al. 2013).

2.4 Condition Assessment of Deteriorating Concrete Structures

Existing concrete structures that show deterioration may need repair or strengthening to achieve their intended service life. In order to perform an effective repair, the cause and extent of damage must be understood, otherwise resources are wasted with an inadequate or unnecessarily expensive repair (Broomfield 2007). This chapter will present common methods used to evaluate the condition of corroding RC structures.

2.4.1 Visual Inspection

Visual inspection of structures is an important part of condition assessment. The intention with a visual inspection is to get a first indication of what the damage is and the extent of it. Starting from a simple general impression of the structure or a part of it, it can end up with the registration of every visible defect on the concrete surface (Bertolini et al. 2013, Broomfield 2007).

2.4.2 Cover Depth Measurement

The measurement of the concrete cover is often combined with a visual inspection. Structures with low concrete covers will be at more risk of corrosion because there, carbonation or chlorides reach the rebars more rapidly. In addition, the availability of oxygen and moisture usually is higher, resulting in a higher corrosion rate. A cover survey is important to explain why some areas of a structure are corroding and to identify areas of future corrosion risk. The cover can be measured with magnetic cover meters, some having the possibility of scanning the surface and obtaining a map of cover depths and rebar layout (Bertolini et al. 2013).

2.4.3 Half-Cell Potential Mapping

Visual examination of structures is an important part of condition assessment. However, damages can only be detected when they become visible on the concrete surface. Electrochemical inspection techniques, such as half-cell potential mapping, can be suitable to investigate the condition of a RC structure at early stages.

Today, potential mapping is the only widely recognised and standardised non-destructive method for assessing the corrosion state of reinforcement bars in concrete structures. With half-cell potential mapping, corrosion can be detected long before it becomes visible at the concrete surface. In addition to being able to locate areas of corrosion for monitoring and condition assessment, it is also useful in determining the effectiveness of repair work (Bertolini et al. 2013).

A corroding reinforcement bar exhibits a potential much more negative than when it is in a passive state. Half-cell potential mapping is based on this difference in potential between corroding and passive steel. A macrocell generates and current flows between the corroding and passive areas of the steel. The electric field, coupled with the corrosion current between corroding and passive areas of the reinforcement bars, can be measured with a suitable reference electrode (half-cell) placed on the concrete surface. This results in a potential field that allows the location of corroding reinforcement bars at the most negative values (Bertolini et al. 2013).

In order to perform the measurement, an electric contact to the reinforcement is established. An external reference electrode is placed on a wet sponge on the concrete surface and potential readings are done with a high-impedance voltmeter on a regular grid on the free concrete surface. The reference electrode is of known potential (half-cell) and all measured potentials are relative to this reference electrode. The copper/copper-sulfate electrode is commonly used and in chloride environments, silver/silver-chloride is also used. By moving the reference electrode across the concrete, different potential values are measured on different locations (Bertolini et al. 2013).

The measured potential varies depending on many factors. If the steel is passive, the measured potential is small (0 to -200 mV). If the passive layer is failing and increasing amounts of steel are dissolving, the potential is typically -350 mV. Corroding steel usually gives potential measurements lower than -350 mV. Very negative potentials can be found in saturated conditions where there is no oxygen to form a passive layer. However, with no oxygen there can be no corrosion, thus this shows the weakness of the potential measurements. It measures the thermodynamics of the corrosion but not the rate of corrosion. The reference electrode potential measurement gives an indication of the corrosion risk of the reinforcement bars. The measurement is linked by empirical comparisons to the probability of corrosion (Broomfield 2007).

The data representation depends on the amount of potential measurements. For small sized elements such as columns, a simple table may be fitting. For large surfaces with thousands of measurements, a better way for presenting the data is a map of the potential field, for example with a colour map where every individual potential measurement can be found as a small coloured square or a map with iso-potential curves. When large areas are to be surveyed, the huge number of potential measurements can be examined statistically (Bertolini et al. 2013).

2.4.4 Concrete Resistivity

The electrical resistivity of concrete is an indicator of the amount of moisture in the concrete pores, and the size and tortuosity of the pore system. It depends on factors such as cement content, water/cement (w/c) ratio, curing, and additives used. Since corrosion is an electrochemical phenomenon, the electrical resistivity of the concrete can be used to draw conclusions about the corrosion performance of a RC structure (Broomfield 2007).

The resistivity of concrete can be measured in various ways. The method for on-site measurements of concrete resistivity involve at least two electrodes, of which one may be a reinforcement bar. A voltage is superimposed between the electrodes, the resulting current is measured and the ratio gives a resistance, measured in Ω (ohms). The resistivity is then obtained by multiplying the measured resistance by a geometrical conversion factor, the so-called cell constant (Bertolini et al. 2013). A modified Wenner probe method is frequently used for the measurement of concrete resistivity on-site. The method includes using a four probe resistivity meter where a current is applied between two outer electrodes and the potential is measured between the two inner. The resistivity for a semi-infinite, homogeneous material can be estimated according to the following equation:

$$\rho = 2\pi a \frac{V}{I} \tag{17}$$

where a is the electrode spacing, I is the applied current across the outer probes and V is the voltage measured between the inner probes (Broomfield 2007).

The interpretation of the resistivity results can be quantitative or qualitative. Measured data can be corrected for the temperature effect and then compared to reference data of similar concrete types. Usually, additional information is necessary. If for example, a wet structure made with ordinary portland cement has a mean resistivity value of 50 Ωm, one can conclude that w/c ratio and the porosity of the concrete must be quite high. Consequently, the concrete will be susceptible to rapid chloride penetration and/or the corrosion rate after the reinforcement bar is depassivated will be high (Bertolini et al. 2013).

2.4.5 Corrosion Rate Measurement

Quantitative information on the corrosion rate of reinforcement bars in concrete is crucial for the assessment of repair methods in the laboratory, for service-life prediction, and structural assessment of corroding structures, as well as control of repair work on-site (Bertolini et al. 2013). Two definitions can be used to describe the corrosion rate: average corrosion rate and instantaneous corrosion rate. The average corrosion rate is used for service life models or for the prediction of degradation of the structure. It can be determined by measuring weight loss or loss of cross section of the steel on-site. The value is taken as the average value over a long period of time. Usually, the time of depassivation is not known, therefore the value will contain an unknown error. The instantaneous corrosion rate, denoted i_{corr}, is determined by using electrochemical methods. They usually involve measuring the polarisation resistance, R_{P} (Bertolini et al. 2013).

Principles of Polarization Resistance Method

The linear polarisation technique (measuring polarisation resistance) requires the polarisation of the steel with an electric current and monitoring the effect on the reference electrode potential. The method is carried out with a sophisticated development of a reference electrode incorporating an auxilary electrode and a variable low voltage DC power supply. The reference electrode potential is measured and then a small current is passed from the auxiliary electrode to the reinforcement. The R_P value is related to the corrosion current i_corr by the equation (Broomfield 2007):

$$i_\mathrm{corr} = \frac{B}{R_\mathrm{P}} \tag{18}$$

where B is a constant that depends on the passivity or active condition of the steel surface and is measured in mV. The polarization resistance R_P is measured in Ω and is given by (Broomfield 2007):

$$R_\mathrm{P} = \frac{\text{change in potential}}{\text{applied current}} \tag{19}$$

For the equation to be valid and remain linear, the change in potential has to be less than 20 mV. For the measurements, either steadily fixed levels of current are applied and the potential is monitored (galvanostatic), or the current is increased in order to achive one or more target potentials (potentiostatic). Change in current versus change in potential are plotted, resulting in a gradient of R_P. This is then used to calculate the steel section loss rate (Broomfield 2007). The corrosion rate is given by the following equation:

$$x = \frac{11 \times 10^6 \cdot B}{R_\mathrm{P} \cdot A} \tag{20}$$

where A is the surface area of steel, measured in cm^2. For accurate corrosion rate measurements, defining the area of measurement is important (Broomfield 2007). The calculation of a corrosion rate is straightforward and valid for general corrosion attack. The calculation of local corrosion rate is difficult since the area of the localised attack is not known (Bertolini et al. 2013).

Execution of Corrosion Rate Measurements

The corrosion rate measurements are slow compared to electrode potential method. Thus it is important to take measurements at the most important parts of the structure. This can be done for example by following up a potential survey with corrosion rate measurements at the positions of the highest and lowest potentials and at the steepest potential gradients (Broomfield 2007).

The corrosion rate varies with the environmental conditions. It is therefore important to take measures at regular intervals throughout the year, or at the same time each year, so that results are comparable (Broomfield 2007).

2.4.6 Carbonation Depth Measurement

Carbonation is easy to detect and measure. Measurements can be taken on concrete cores, fragments, and down drilled holes. A pH indicator, usually an alcoholic solution of phenolphthalein, will detect the change in pH across a freshly exposed concrete face. The areas with pH > 9 take on a pinkish color in a basic environment, while the color of carbonated areas remains unchanged (Bertolini et al. 2013, Broomfield 2007).

Carbonation depth sampling allows the average and standard deviation of the carbonation depth to be calculated. Comparing this value to the average reinforcement cover, the amount of depassivated reinforcement steel can then be estimated. In addition, if the carbonation rate can be determined from laboratory testing and historical data, then the progression of depassivation with time can be calculated (Broomfield 2007).

2.4.7 Chloride Content Determination

Chlorides in concrete structures are usually measured by producing chloride depth profiles. This is essential because knowing the chloride level at the reinforcement determines the present extent of corrosion, while the chloride profile determines the future development of corrosion (Broomfield 2007).

Several methods can be used to measure chloride content. On-site methods are quick but less accurate, while laboratory methods are more accurate but more time consuming and costly. Most methods require sampling, crushing to powder, dissolution in acid, and chloride analysis. Recently, grinding powder for slices of one or two millimeter thickness has become widely used for measuring accurate penetration profiles. Simple methods based on spraying the surface of split cores with silver nitrate, where a colour change indicates the chloride penetration front, have also been proposed (Bertolini et al. 2013).

The obtained chloride profiles give information about the transport of chlorides into the concrete. Combining them with results from potential mapping, the critical chloride content for a specific structure can be found. An empirical correlation between the chloride content and half-cell potential can be established, consequently a roughly estimated distribution of chlorides can be obtained from the potential map (Bertolini et al. 2013).

2.4.8 Corrosion Monitoring

In order to gain understanding of the development of the condition of a bridge, it is necessary to monitor the condition change with time. In existing structures, monitoring can be done by regular inspection using the techniques previously described, or by the installation of embedded sensors that are built in the structure either at the time of construction or inserted later (Bertolini et al. 2013, Broomfield 2007).

Long-term monitoring brings obvious advantages. By detecting corrosion risk conditions for the reinforcement early, it can be sufficient to implement relatively simple maintenance measures or protection systems in a cost-effecive way and damage can be avoided. In addition, monitoring the progress of condition changes allows to predict the future development of the structure which can be used as a basis to determine the optimal time of an intervention (Bertolini et al. 2013).

2.5 Repair and Rehabilitation Methods

Concrete structures that have suffered from reinforcement corrosion need to be repaired in order to reach their expected service life. The recent The recent Model Code of the International Federation for Structural Concrete *(fib)* proposes maintenance procedures to be included both within proactive and reactive approaches. Proactive maintenance measures prevent damage before it becomes critical for the required performance of the structure. Proactive maintenance measures are usually regular and planned. Reactive maintenance measures refers to repairs that are done when damages have already taken place. This is the only possibility for existing structures that were not subject to specific durability design at the time of construction and are now in the propagation phase of corrosion (Bertolini et al. 2013).

The first step in the process of repair and rehabilitation is a thorough condition analysis of the structure, where the following is evaluated:

- Causes of reinforcement corrosion;
- The extent of damage;
- The expected progress of damage;
- The influence of the damage on the serviceability and structural safety of the structure (Bentur et al. 2005).

On the basis of the assessment, the optimal strategy for repair and maintenance is selected. The decision on the repair option should be based on the extent and cause of damage, if damage is to be expected, the intended use and importance of the structure and the consequences of degradation for its structural safety and serviceability (Bertolini et al. 2013). The strategy chosen should also take into account economic aspects, such as the cost of the repair options, and the additional service life the structure is expected to serve (Bentur et al. 2005).

A decision could for example be made to replace or partially reconstruct components that are severely damaged. Often, methods to stop corrosion or reduce the corrosion rate in an existing structure are adopted, sometimes even after the repair or replacement of a damaged concrete. In some cases, if the extent of damage is limited or the remaining service life is short, it may be the best solution to not intervene, but keep the corrosion and serviceability under control by some form of monitoring. In some cases, if the structural requirements are not fulfilled, the function of the structure may be downgraded (Bertolini et al. 2013).

2.5.1 Repair Principles

The corrosion rate of the reinforcement can be controlled or reduced by stop the anodic process or the transport of current within the concrete (electrolytic process). These methods are based on different principles, shown in fig. 11, along with several different repair techniques that can be used.

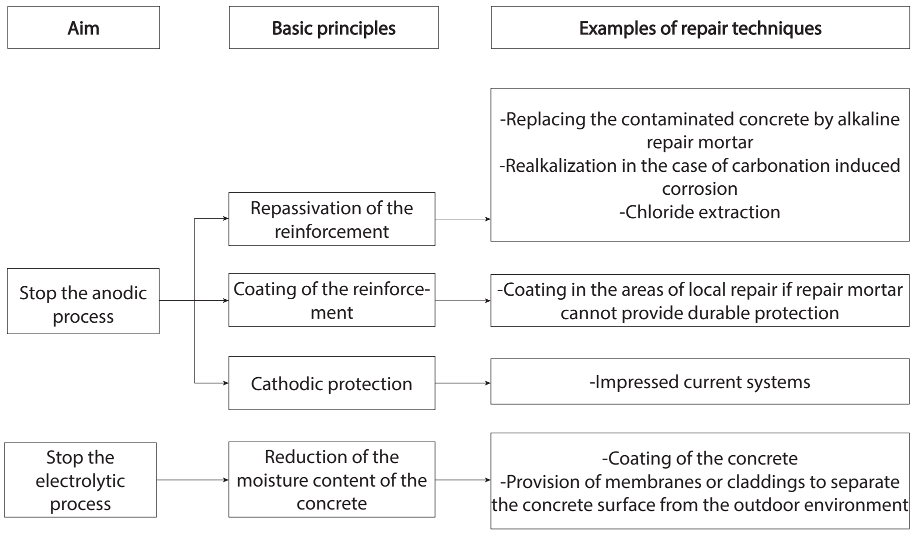

Figure 11: Principles of repair for damaged concrete due to corrosion. Based on (Bertolini et al. 2013).

Various repair methods are available for structures that have been damaged by carbonation induced or chloride induced corrosion. Bertolini et al. (2013) summarised the methods used for repair of carbonated structures and chloride-contaminated structures, which are listed below.

2.5.2 Repair Methods for Carbonated Structures

Repassivation Methods

Repassivation methods are based on the repassivation of reinforcement bars. For structures that have suffered carbonation induced corrosion, this means that the alkaline conditions have to be restored around the steel reinforcement bars.

Conventional repair removal of concrete and its replacement with alkaline mortar or concrete. This method is convenient if the corrosion attack is limited to small zones, for example when the concrete cover thickness is reduced locally. This type of repair can be expensive if the area removed is large. All concrete where carbonation and subsequent reinforcement corrosion are expected to damage the structure within the design life of the repair should be removed. Surface coating may be applied on the repaired surface and the non-repaired surface to increase the resistance to carbonation. When the extent of carbonation is large, it can be necessary to remove a large amount of mechanically sound concrete. To avoid this costly repair, other methods have been developed to realkalise the original concrete;

Repassivation with Alkaline Concrete or Mortar the application of a adequately thick cement-based layer of concrete or mortar over the surface of carbonated concrete can induce realkalization of the underlying layer. Cracked or delaminated concrete has to be removed, while sound and even carbonated concrete does not. The method relies on the diffusion of hydroxyl ions from the new external alkaline layer towards the underlying carbonated concrete layer. This can occur in wet environments or in the presence of wetting–drying cycles, and can lead to the repassivation of the reinforcement. If the carbonation has penetrated behind the reinforcement more than 20 mm it should not be used;

Electrochemical Realkalization involves the application of direct current to the reinforcement from a temporary anode placed on the surface of the concrete. The repair lasts several days up to a few weeks. The realkalization of concrete takes place both from the surface of the concrete and from the surface of the steel;

Cathodic Protection this method requires the permanent application of small current to the steel. Can lead to repassivation of the reinforcement because of the realkalisation of the concrete around the steel. This method is usually applied for chloride induced corrosion, but has been used in carbonated structures where small amounts of chloride are present.

Moisture Content Reduction of Concrete

In a dry concrete (and at least in the absence of chloride contamination), even if carbonation has reached the reinforcement, the corrosion rate is very low. In high humidity environments, or environments where wetting-drying cycles occur in the concrete, the application of surface protection that avoid moisture absorption of water from the environment may lead to a lower moisture content in the concrete. Consequently, the reinforcement corrosion rate lowers. Hydrophobic treatments, impermeable coatings, or cladding systems are examples of methods where this can be achieved. These methods are however temporary, and the coatings need to be maintained or even replaced. Surface coatings can also be used to restrict carbon dioxide or chloride penetration in zones where corrosion has not yet initiated.

Coating of the Reinforcement

By applying organic coating, preferably epoxy based, to the reinforcement the anodic process can be stopped. That is because the coating acts as a physical barrier between the steel and repair mortar. The protection is only based on the barrier, and the passivation of steel cannot be achieved because the steel is in no contact with the alkaline repair material. This method should only be used as last resort technique.

2.5.3 Repair Methods for Chloride Contaminated Structures

The process of stopping or reducing the corrosion rate in RC structures that have suffered damage due to chloride induced corrosion is more challenging than in carbonated concrete. That is, among other reasons, because it is difficult to define a critical chloride content due to it being dependent on the concrete composition and exposure conditions.

Repassivation Methods

Repassivation of reinforcement steel in chloride-contaminated structures can be achieved by replacing the concrete with chloride free material (conventional repair), by removing chlorides from the concrete (electrochemical chloride removal), or by cathodic protection.

Repassivation with Alkaline Mortar or Concrete replacement of chloride-contaminated concrete with chloride-free and alkaline mortar or concrete. In addition, all chloride-contaminated rust around the reinforcement and in pits must be removed. Due to the mechanism of chloride induced corrosion, it is not sufficient to only repair the concrete in the area where the reinforcement is depassivated. The concrete has to be removed in all areas where the chloride threshold value has reached the depth of the reinforcement, or where it is expected to reach it during the design life of the repair. The concrete that surrounds the zones of corrosion usually has a chloride content higher than the chloride threshold, even though the steel remains passive because it is protected by the corroding site. It is not enough to only replace the concrete near the corroded area, because corrosion might start in the surrounding areas instead. The use of sacrifical anodes embedded in the repair patches has been proposed, which could prevent corrosion in the reinforcement bars that surround the repaired area;

Electrochemical Chloride Removal involves the application of direct current to the reinforcement from a temporary anode placed on the surface of concrete. The repair can last up to several weeks. Chloride ions are removed from the concrete since they migrate towards the surface of concrete due to the applied current;

Cathodic Protection same method as desribed for carbonated structures. Cathodic protection can stop corrosion for any level of chloride contamination of concrete or exposure condition of the structure, if applied properly.

Other Methods

Repassivation methods should be preferred for chloride-contaminated structures due to the high penetration rate of pitting corrosion and uncertainty due to structural consequences of localised attacks. In the case of low chloride content in concrete and the penetration of chloride is limited, other repair methods can be considered.

Hydrophobic Treatment can be used to control the corrosion rate because it reduces the moisture content of concrete. However, there is insufficient knowledge about the critical moisture content in the case of chloride induced corrosion;

Coating of the Reinforcement applying the same method of coating of reinforcement as described earlier regarding repair methods for carbonated structures, is difficult in chloride-contaminated concrete and should only be considered when other repair techniques are unavailable. It is necessary to remove concrete around the corroding reinforcement or where it is expected to corrode during the design life of the repair, and to remove all chlorides from pits. In order to insure adhesion between the coating and the reinforcement, the surface to be treated must be carefully prepared. This can be difficult in practise;

Migrating Inhibitors are substances that, once applied on the concrete surface, can migrate through the concrete cover. The effect of this occurs only if the inhibitor blend reaches unaltered the reinforcement surface.

2.6 Summary

Steel corrosion is one of the biggest durability issues for RC structures. Concrete is alkaline in nature which results in a protective, passive oxide layer to form on the surface of the reinforcing steel that reduces the corrosion rate to a very low, insignificant rate. For the steel to corrode, the passive layer must be destroyed.

The service life of a concrete structure can be divided into two phases, the initiation phase and the propagation phase. During the initiation phase, substances such as carbon dioxide and chlorides, penetrate into the concrete cover and can depassivate the steel. The propagation phase begins when the steel is depassivated. In this phase, the reinforcement is corroding, which can lead to the deterioration of the concrete as well.

The corrosion process is electrochemical and involves two half-cell reactions, anodic reaction and cathodic reaction. From the corrosion process, rust is formed, which involves a substantial volume increase. The corrosion rate determines the time it takes to reach an undesirable event. It is usually expressed as the penetration rate and is measured in μm/year. The two main forms of corrosion are general corrosion and pitting corrosion. General corrosion is a typical characteristic of a carbonation induced corrosion while pitting corrosion is typical for chloride induced corrosion.

Very high penetration rates can be reached in the event of pitting corrosion which can reduce the cross sectional area of the steel quickly, without having signs of cracking or spalling, making it difficult to identify during visible inspections.

There are two mechanisms that will cause the passive layer to depassivate: chloride ions and carbonation. Carbonation involves a process that lowers the pH level of the concrete, which causes the reinforcement steel to corrode. The rate of carbonation roughly follows Fick's law of diffusion. For chloride induced corrosion there is no overall drop in the pH level of concrete but instead the chlorides help to break down the protective oxide layer and allow the corrosion process to proceed quickly. A threshold value, C_{crit}, is necessary to sustain local passive film breakdown. Experience shows that chloride profiles can be estimated by Fick's second law of diffusion. Both carbonation and chloride induced corrosion are dependent on several material and environmental factors.

Reinforcement corrosion causes degradation of RC structures. The damages include cracks, spalling or complete delamination, reduction in steel diameter and cross sectional area, and so on. These damages can affect the appearance, serviceability, safety, and structural performance of the structure.

To evaluate the condition of corroding RC structures, different methods are available. In addition, a good way to understand the development of the condition of a structure is to monitor the condition change with time, for example with built-in sensors.

In order to decide on a repair method, the cause, extent, expected development, and the influence of a damage needs to be evaluated. On the basis of the assessment, the optimal repair and maintenance strategy is chosen. Various repair methods are available for structures that have been damaged by carbonation induced or chloride induced corrosion.

3 Decision Theory

3.1 General

In this chapter the analysis of decision making when uncertainty is involved will be presented. An imaginary example is used to illustrate decision analysis for the selection of an inspection strategy for a bridge suffering from damage. In practise, this decision analysis method can be used for all kinds of decisions, for example what type of inspection, repair, or maintenance method to choose from, the extent of measurements, comparing damages, prioritisation between different bridges, and so on.

3.2 Knowledge and Uncertainty

In all fields of engineering there exists some degree of uncertainty. This uncertainty can for example be associated with the geometry of a structure, material properties, loading, and environmental conditions (JCSS 2001). During a bridge inspection and assessment, the procedures depend heavily on different parameters such as the type and use of structure, implied risks and/or costs, the condition of the structure, and so forth. The models used in design and assessment of structures therefore reflect the imperfect knowledge about the structure (JCSS 2001).

The reassessment procedure of a structure is a decision process with the purpose of identifying the measures which will lead to the most economical fulfilment of specified requirements regarding present and future use of the structure. Such measures can for example be the inspection and gathering of information regarding the geometry of the structure, the material properties, the loading, the degree of deterioration of the structure, or the behaviour of the structure. It can also be the repair, strengthening, or changing of the structures intended use. It is necessary to evaluate the measures in terms of their consequence on safety and monetary value throughout the required service life (JCSS 2001).

3.3 Decision Making Under Uncertainty

Decision making involves, in many cases, uncertainty in the elements of the decision process. This introduces the possibility of the decision made being sub-optimal, in the sense that better outcomes could have been achieved with a different decision. By acquiring more information, uncertainty can be reduced, but it can be costly (Fenwick et al. 2020). In some cases, it might be feasible but not economical to obtain additional information regarding the state in question (Benjamin & Cornell 1970). Before resources are invested in gathering additional information, the associated costs and benefits should be considered (Fenwick et al. 2020).

3.4 Bayesian Decision Analysis

Decision analysis is an approach developed in the 1960s, where the elements of a decision making process are quantified in order to determine the optimal decision. Raiffa & Schlaifer (1961) present a consistent framework, Bayesian Decision Analysis, that accounts for uncertainty and additional information in decision making. There, they introduce the mathematical analysis of decision making when the state of nature is uncertain, but additional information about it can be obtained by experiments. In this setting, an experiment has a wide meaning and can also for example be a computer based analysis, such as reliability analysis or finite element method analysis. The objective of a decision analysis is to identify a course of action, which may or may not include an experiment, that is logically consistent with the decision maker's own preferences for consequences.

The simplest form of the decision analysis is a priori analysis. In this analysis, a decision maker has to choose among actions a, with the possible states of nature θ and their assigned prior probabilities $P'(\theta)$. The expected utility $u(a, \theta)$ of each action-state pair are evaluated and maximised subject to a. A posterior analysis is in principle of the same type as a priori decision analysis, but here the additional information about the state becomes available, which is represented as the outcome of an experiment z. This additional information is expressed by the likelihood $P(z \mid \theta)$. The posterior probabilities $P''(\theta)$ are obtained by combining the likelihood with the prior probability, $P''(\theta \mid z) \propto P(z \mid \theta)P'(\theta)$. The utility to be maximised subject to a is then also conditional on z, $u(a, \theta \mid z)$. The priori and posterior analysis are illustrated by a simple decision tree in fig. 12.

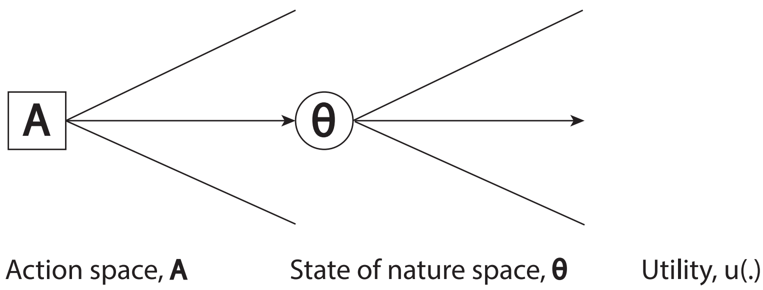

Figure 12: Decision tree to illustrate priori- and posterior decision analysis. Based on (Fenwick et al. 2020).

A preposterior analysis describes the value of performing an experiment e and the possible corresponding outcome z, in order to reduce the expected costs. The analysis examines whether or not the money spent for acquiring additional information has been cost efficient. The outcomes of an experiment are weighted according to prior probabilities $P'(\theta)$. The utility $u(e, z, a, \theta)$ can be maximised subject to e and a. A preposterior analysis is illustrated by a simple decision tree in 13.

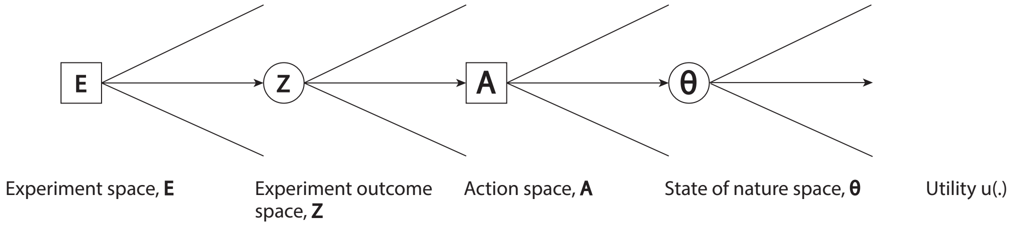

Figure 13: Decision tree to illustrate preposterior decision analysis. Based on (Fenwick et al. 2020).

3.5 Decision Problem

During a bridge assessment, decisions needs to be made regarding the bridge maintenance. Various inspection and repair options are available for the evaluation of the actual condition and for the repair of bridges. For the purpose of illustration, a hypothetical example where the process of selecting the optimal inspection and repair is presented. The example is based on hypothetical probabilities, costs, and consequences. The theoretical background of the example is based on Benjamin & Cornell (1970). In appendix A, a number of key ideas from probability theory are presented.

3.6 Priori Analysis

3.6.1 Decision Model

In a decision making process, there are several available actions $a_0, a_1, ..., a_{\mathrm{n}}$ to choose from an action space A. In the illustrative example, the possible actions are to repair or not repair the structural element:

a_0: repair;

a_1: no repair.

The state of nature space, denoted Θ, includes all possible events or states of nature $\theta_0, \theta_1, ..., \theta_{\mathrm{n}}$. Only one state is the true state. In the example, there are two possible discrete states of nature, damage or no damage of the structural element:

θ_0: damage;

θ_1: no damage.

The possible states have assigned probabilities $P'(\theta_{\mathrm{i}})$, that is, the probability that the true state of nature is θ_{i}. In general, these probabilities are often based on prior experience and information available. In this example, the probability of damage, $P'(\theta_0)$, increases with time, as shown in fig. 14. The probability of failure for each year considered in this analysis is to be found in table 19 in appendix D.1. Probability of no damage, $P'(\theta_1)$ is equal to $1 - P'(\theta_0)$.

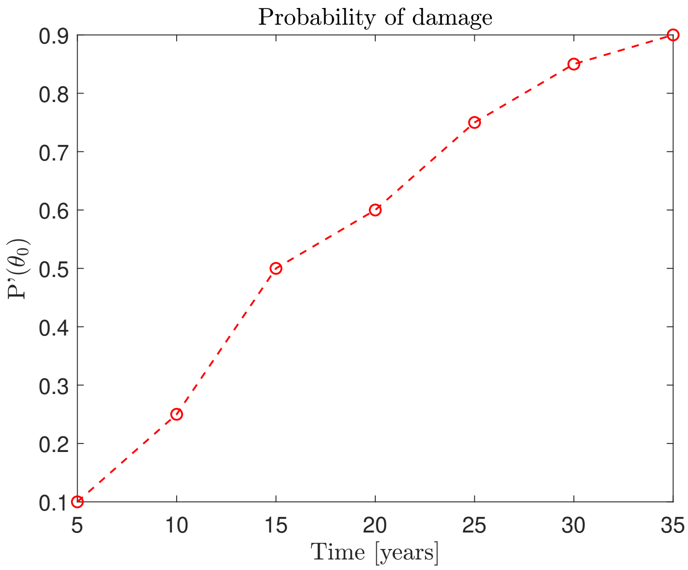

Figure 14: Probability of damage, $P'(\theta_0)$, as a function of time.

A decision tree for the priori analysis is shown in fig. 15. The decision tree shows the actions a_0 and a_1, and the states of nature θ_0 and θ_1 and their assigned probabilities.

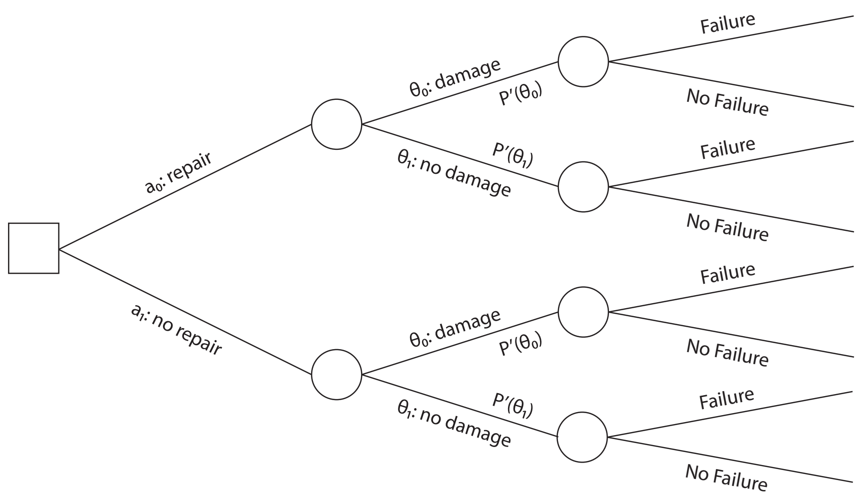

Figure 15: A priori decision tree.

If the bridge is repaired, the probability of failure is the same whether there is or is not damage before the repair, $P(F \mid a_0) = P(F \mid \theta_0 \cap a_0) = P(F \mid \theta_1 \cap a_0)$. The probability of failure given that no repair has been performed, changes with time, as the structure degrades with time. This probability, denoted $P(F \mid a_1)$, can be found with eq. (21):

$$P(F \mid a_1) = P'(\theta_0) \cdot P(F \mid \theta_0 \cap a_1) + P'(\theta_1) \cdot P(F \mid \theta_1 \cap a_1) \tag{21}$$

where F indicates failure. 3 conditional probabilities are assumed until next inspection of the bridge:

$\boldsymbol{P(F \mid a_0) = 10^{-6}}$ given that a repair is performed, the probability of failure for the existing bridge is 10^{-6};

$\boldsymbol{P(F \mid \theta_0 \cap a_1) = 10^{-3}}$ given a damage to the bridge and no repair is performed, the probability of failure for the existing bridge is 10^{-3};

$\boldsymbol{P(F \mid \theta_1 \cap a_1) = 10^{-6}}$ given no damage to the bridge and no repair is performed, the probability of failure for the existing bridge is 10^{-6}.

For each action-state pair the probability of no failure is $P(F^C) = 1 - P(F)$. Once a decision has been made, either action a_0 or a_1, the true state will either be θ_0 or θ_1. As a result, the utility $u(\theta, a)$, measuring the consequences of the action-state pair, must be quantified. The cost of repair is assumed to be $C_{\mathrm{R}} = 30{,}000$ NOK and the cost of failure, $C_{\mathrm{F}} = 45$ MNOK, including all costs related to the reconstruction of the bridge.

The expected utility for each outcome is calculated in the following way:

$$E[u \mid \theta_0 \cap a_0] = P(F \mid \theta_0 \cap a_0) \cdot C_{\mathrm{F}} \tag{22}$$

$$E[u \mid \theta_1 \cap a_0] = P(F \mid \theta_1 \cap a_0) \cdot C_{\mathrm{F}} \tag{23}$$

$$E[u \mid \theta_0 \cap a_1] = P(F \mid \theta_0 \cap a_1) \cdot C_{\mathrm{F}} \tag{24}$$

$$E[u \mid \theta_1 \cap a_1] = P(F \mid \theta_1 \cap a_1) \cdot C_{\mathrm{F}} \tag{25}$$

The expected value of action a_0 is:

$$\begin{aligned} E'[u \mid a_0] &= P'(\theta_0) \cdot u[\theta_0 \mid a_0] + P'(\theta_1) \cdot u[\theta_1 \mid a_0] + C_{\mathrm{R}} \\ &= u[\theta_0 \mid a_0] + C_{\mathrm{R}} \\ &= P(F \mid a_0) \cdot C_{\mathrm{F}} + C_{\mathrm{R}} \end{aligned} \tag{26}$$

The expected value of action a_1 is:

$$\begin{aligned} E'[u \mid a_1] &= P'(\theta_0) \cdot u[\theta_0 \mid a_1] + P'(\theta_1) \cdot u[\theta_1 \mid a_1] \\ &= [P'(\theta_0) \cdot P(F \mid \theta_0 \cap a_1) + P'(\theta_1) \cdot P(F \mid \theta_1 \cap a_1)] \cdot C_{\mathrm{F}} \\ &= P(F \mid a_1) \cdot C_{\mathrm{F}} \end{aligned} \tag{27}$$

The expected value from the priori analysis is:

$$E'[u] = \min(E'[u \mid a_0], E'[u \mid a_1]) \tag{28}$$

3.6.2 Result of Priori Analysis

The probability of failure given that no repair has been performed is calculated according to eq. (21), and the expected values for action a_0 and a_1 according to eq. (26) and eq. (27), respectively. The calculation results are given in table 19 in appendix D. The expected values for each action from the priori analysis, along with the expected value from the priori analysis found with eq. (28), are given in table 3 and illustrated graphically in fig. 16.

Table 3: Expected values from the priori analysis.

Year	$\boldsymbol{E'[u \mid a_0]}$ **[NOK]**	$\boldsymbol{E'[u \mid a_1]}$ **[NOK]**	$\boldsymbol{E'[u]}$ **[NOK]**
5	30,045	4,541	4,541
10	30,045	11,284	11,284
15	30,045	22,523	22,523
20	30,045	27,018	27,018
25	30,045	33,761	30,045
30	30,045	38,257	30,045
35	30,045	40,505	30,045

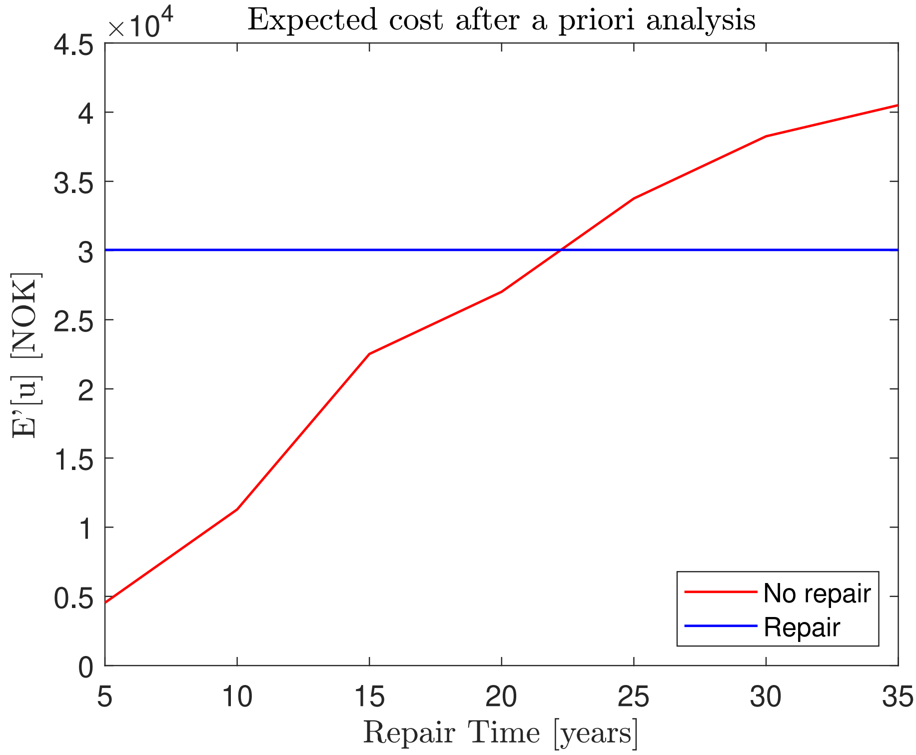

Figure 16: Comparison of expected costs for priori analysis of action a_0 and a_1.

From the priori analysis, action a_1, performing no repair, is the most economical decision for the first 20 years, after which it becomes more economical to perform the repair. That can be explained by the fact that if the bridge is not repaired, the probability of failure will become higher every year, consequently making expected failure costs higher every year. At and after year 25, the probability of failure is so high that the expected failure cost is higher than the cost of repair.

3.7 Posterior Analysis

3.7.1 Additional Information

Additional information about the state of the bridge or bridge element can be provided by performing an inspection. The inspection space, E, consists of all available inspection strategies that can be performed. Two types of inspections are being considered in this example:

- e_1: visual inspection;
- e_2: sensor inspection.

As a result of the inspections, two possible indications in the inspection outcome space Z are observed. They are:

- z_0: damage is detected;

- z_1: damage is not detected.

3.7.2 Updating of Damage Probabilities

The two inspection methods differ in their accuracy. This accuracy is quantified by the probability of detection or no detection, given that the structure is either subject to damage or not. The sensor inspection is considered to be more accurate, thus having higher probability of detection of damage. The accuracy of the inspection methods are given in table 4 and table 5.

Table 4: Accuracy of visual inspection e_1.

Inspection outcomes Z	$\boldsymbol{\theta_0}$ **: damage**	$\boldsymbol{\theta_1}$ **: no damage**
$\boldsymbol{z_0}$ **: damage is detected**	$P(z_0 \mid \theta_0) = 0.80$	$P(z_0 \mid \theta_1) = 0.25$
$\boldsymbol{z_1}$ **: damage is not detected**	$P(z_1 \mid \theta_0) = 0.20$	$P(z_1 \mid \theta_1) = 0.75$

Table 5: Accuracy of sensor inspection e_2.

Inspection outcomes Z	$\boldsymbol{\theta_0}$ **: damage**	$\boldsymbol{\theta_1}$ **: no damage**
$\boldsymbol{z_0}$ **: damage is detected**	$P(z_0 \mid \theta_0) = 0.90$	$P(z_0 \mid \theta_1) = 0.05$
$\boldsymbol{z_1}$ **: damage is not detected**	$P(z_1 \mid \theta_0) = 0.10$	$P(z_1 \mid \theta_1) = 0.95$

Based on additional information and prior probabilities $P'(\theta_\mathrm{i})$, Bayes' rule, given by eq. (64) in appendix A, can be applied to compute posterior probabilities $P''(\theta_\mathrm{i})$:

$$P''(\theta_\mathrm{i}) = P(\theta_\mathrm{i} \mid z_\mathrm{k}) = \frac{P(z_\mathrm{k} \mid \theta_\mathrm{i}) \cdot P'(\theta_\mathrm{i})}{\sum_\mathrm{j} P(z_\mathrm{k} \mid \theta_\mathrm{j}) \cdot P'(\theta_\mathrm{j})} \tag{29}$$

Once the posterior probabilities have been calculated, the decision analysis continues with the same procedure as in the priori analysis.

3.7.3 Result of Posterior Analysis

Given detection of damage z_0, the posterior probabilities and expected values for actions a_0 and a_1 were found, see table 20 (visual inspection) and table 21 (sensor inspection) in appendix D.2. The expected values from the posterior analysis for visual inspection e_1 are found in table 6, and for sensor inspection e_2 in table 7:

Table 6: Expected values for visual inspection - given detection.

Year	$E[u \mid a_0]$ [NOK]	$E[u \mid a_1]$ [NOK]	$E[u, e_1]$ [NOK]
5	30,045	11,836	11,836
10	30,045	23,248	23,248
15	30,045	34,296	30,045
20	30,045	37,249	30,045
25	30,045	40,759	30,045
30	30,045	42,650	30,045
35	30,045	43,491	30,045

Table 7: Expected values for sensor inspection - given detection.

Year	$E[u \mid a_0]$ [NOK]	$E[u \mid a_1]$ [NOK]	$E[u, e_2]$ [NOK]
5	30,045	30,015	30,015
10	30,045	38,578	30,045
15	30,045	42,634	30,045
20	30,045	43,394	30,045
25	30,045	44,183	30,045
30	30,045	44,564	30,045
35	30,045	44,724	30,045

The results in table 6 show that given detection of damage by visual inspection, it becomes more economical to perform the repair earlier than indicated in the priori analysis, or at year 15, because that is when action a_1 starts to have higher costs than action a_0. Same goes for the sensor method, as can be seen in table 7, it becomes more economical to repair the damage even earlier, or at year 10. Additionally, it can be observed that the expected costs for action a_0 and a_1 at year 5 are approximately the same, meaning it would for the most part lead to the same conclusion if the damage is repaired from the beginning. The expected costs for visual and sensor inspection given detection, are compared in fig. 17.

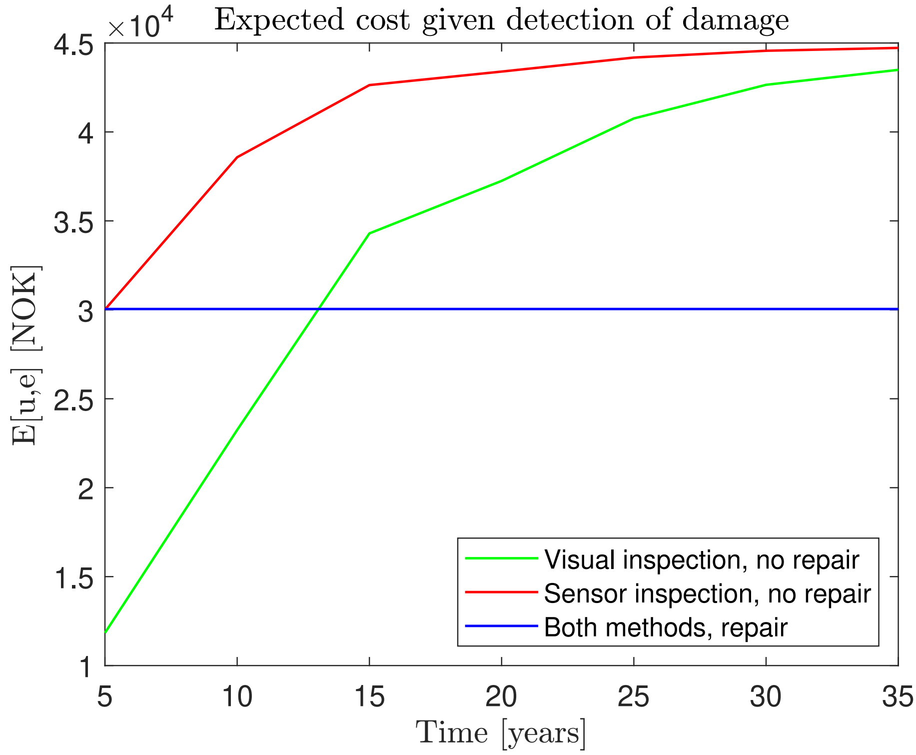

Figure 17: Expected cost given detection of damage.

Given no detection of damage z_1, the posterior probabilities and expected values for actions a_0 and a_1 were calculated and can be found in table 22 (visual inspection) and table 23 (sensor inspection), in appendix D.2. The expected values from the posterior analysis for visual inspection e_1 are found in table 8 and for sensor inspection e_2 in table 9:

Table 8: Expected values for visual inspection - given no detection.

Year	$E[u \mid a_0]$ [NOK]	$E[u \mid a_1]$ [NOK]	$E[u, e_2]$ [NOK]
5	30,045	1,339	1,339
10	30,045	3,715	3,715
15	30,045	9,509	9,509
20	30,045	12,889	12,889
25	30,045	20,025	20,025
30	30,045	27,098	27,098
35	30,045	31,778	30,045

Table 9: Expected values for sensor inspection - given no detection.

Year	$\boldsymbol{E[u \mid a_0]}$ **[NOK]**	$\boldsymbol{E[u \mid a_1]}$ **[NOK]**	$\boldsymbol{E[u, e_2]}$ **[NOK]**
5	30,045	565	565
10	30,045	1,569	1,569
15	30,045	4,326	4,326
20	30,045	6,175	6,175
25	30,045	10,834	10,834
30	30,045	16,841	16,841
35	30,045	21,915	21,915

The results in table 8 show that given no detection of damage by visual inspection, it becomes economical to perform the repair late in the structure's lifetime, or at year 35. For the sensor inspection, it is economical to not perform the repair during the time period considered, as shown in table 9. The results are shown in fig. 18.

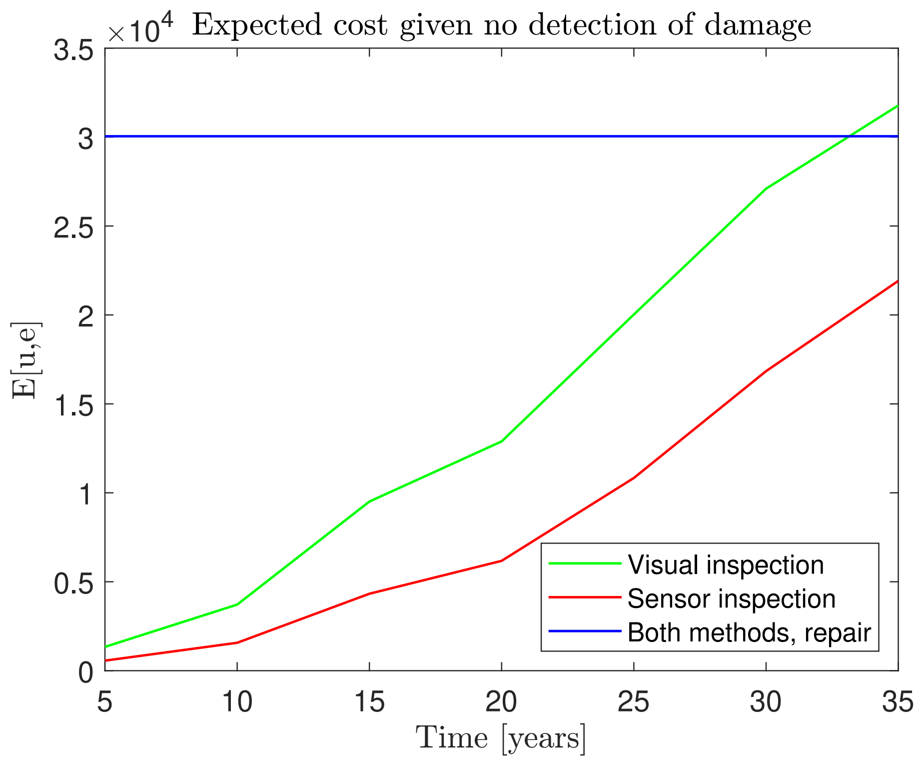

Figure 18: Expected cost given no detection of damage.

3.8 Preposterior Analysis

3.8.1 Inspection Optimisation

A preposterior analysis is about examining whether or not the funds spent for acquiring additional information, for example through an inspection or an experiment, will be or has been cost efficient. Often, a decision maker has the option to pay to observe the outcome of an inspection before choosing an action a. If the cost of an inspection is low compared to the information it brings on the state of nature θ, the inspection should be chosen. If several inspections are considered, the one that brings the best balance between inspection cost and reduced risk in the choice of action should be chosen.

3.8.2 Cost of Inspection

The cost of inspection method e_1 is C_{e_1} = 3,000 NOK and the cost of inspection method e_2 is $C_{\mathrm{e}_2} = 9,000$ NOK. No inspection, which hereby is denoted e_0, has no cost, $C_{\mathrm{e}_0} = 0$. This cost will now be added to all the action-state pairs for each corresponding inspection method.

3.8.3 Utility Formulas

Having chosen an inspection e, an outcome z from the inspection outcome space Z will be observed. Then the decision maker has to choose an action a from the action space A, after which the true state of nature θ from the state of nature space Θ will be found. As a result, the utility $u(e, z, a, \theta)$ will be found. For every possible experiment-outcome pair, a posterior analysis is made.

The posterior expected utilities are:

$$E[u(e, z, a) \mid e, z] = \sum_{\mathrm{i}} u(e, z, a, \theta_{\mathrm{i}}) \cdot P(\theta_{\mathrm{i}} \mid e, z) \tag{30}$$

where $P(\theta_{\mathrm{i}} \mid e, z) = P''(\theta_{\mathrm{i}})$, except in the case of no inspection, where $P(\theta_{\mathrm{i}} \mid e, z) = P'(\theta_{\mathrm{i}})$. The expected utility for each inspection-outcome pair is then found by:

$$u(e, z) = \max_{\mathrm{a}} [E[u(e, z, a) \mid e, z]] \tag{31}$$

The expected utility of each inspection is:

$$E[u(e)] = \sum_{\mathrm{k}} u(e, z_{\mathrm{k}}) \cdot P(z_{\mathrm{k}} \mid e) \tag{32}$$

where the probability of an outcome given the inspection e is:

$$P(z_{\mathrm{k}}) = P(z_{\mathrm{k}} \mid e) = \sum_{\mathrm{i}} P(z_{\mathrm{k}} \mid e, \theta_{\mathrm{i}}) \cdot P'(\theta_{\mathrm{i}}) \tag{33}$$

3.8.4 Preposterior Decision Tree

As for the priori analysis, a decision tree can be established for the preposterior analysis. The preposterior decision tree is shown in fig. 19.

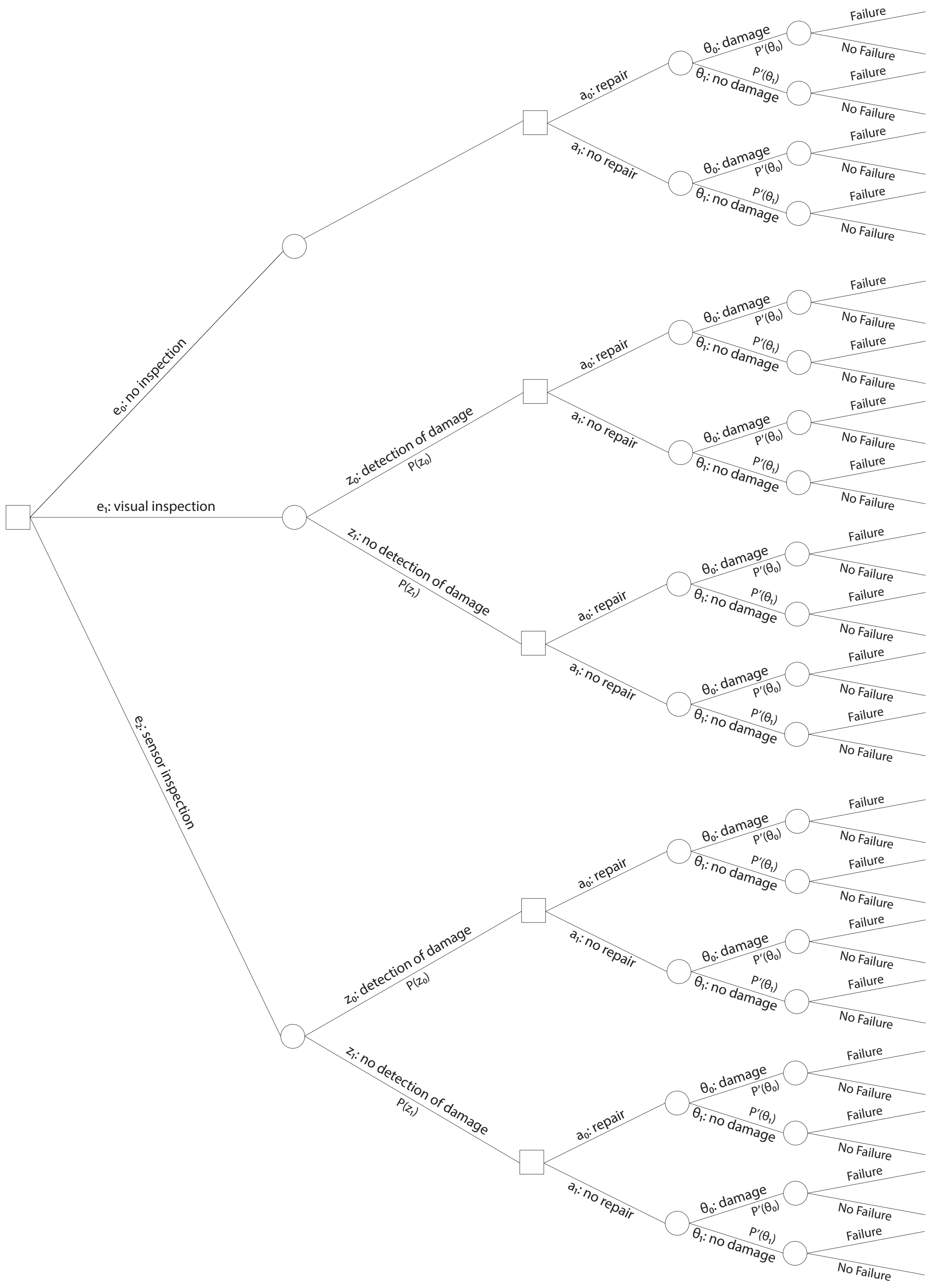

Figure 19: A posterior decision tree.

3.8.5 Results of Preposterior Analysis

Since no inspection e_0 does not involve any cost or detection probability, the results for e_0 are the same as in the priori analysis found in section 3.6. The posterior expected utilities for inspection methods e_1 and e_2, found by eq. (30), are given in table 24 for inspection method e_1, and in table 25 for inspection method e_2, which are found in appendix D. Additionally in appendix D are the outcome probabilities, expected utilities for each inspection-outcome pairs, and the expected utility of the inspection, in table 26 for inspection method e_1, and in table 27 for inspection method e_2. The expected costs for each inspection method is shown in table 10 and illustrated in fig. 20.

Table 10: Expected costs for inspection methods e_0, e_1, and e_2.

Year	$E[u, e_0]$ [NOK]	$E[u, e_1]$ [NOK]	$E[u, e_2]$ [NOK]
5	4,541	7,541	13,541
10	11,284	14,284	18,044
15	22,523	23,291	25,543
20	27,018	25,840	28,542
25	30,045	29,663	33,042
30	30,045	32,212	36,041
35	30,045	33,045	37,541

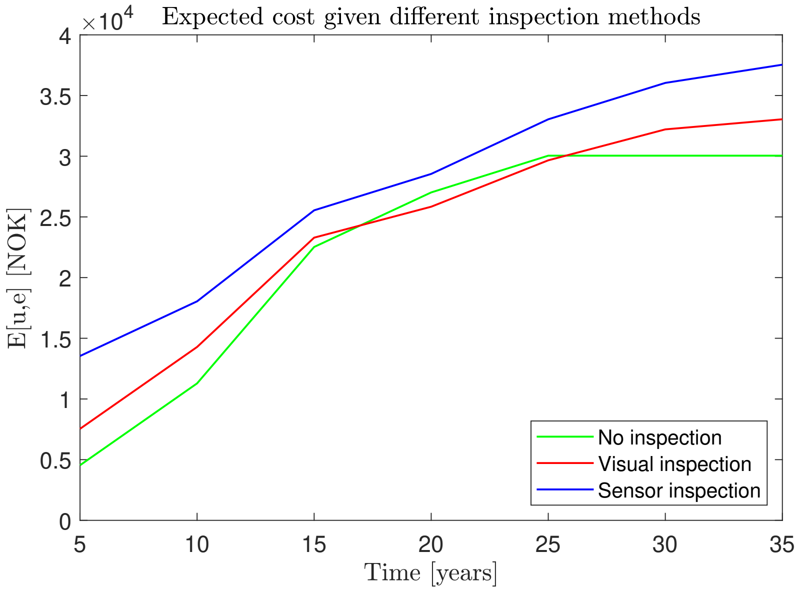

Figure 20: Expected costs for inspection methods e_0, e_1, and e_2.

As can be seen in table 10 and fig. 20, inspection method e_2 gives a higher expected cost than not doing any inspection e_0 in each case. Due to its high cost, inspection method e_2 is not recommended over doing no inspection.

Performing no inspection has the lowest expected cost during the first 15 years. At years 20 and 25, the expected cost of performing no inspection, e_1, is higher than performing visual inspection. Then, at year 30-35, the expected cost of no inspection goes back to being lower than the expected cost of inspection method e_1.

Based on this information, it is therefore recommended to not perform any inspection during the first 15 years. At years 20 and 25, it is recommended to perform a visual inspection. At years 30-35, it is recommended to not perform any inspection.

3.8.6 Value of Information

An inspection e_i is considered to be economical if the cost of it is lower than its value of information ($VoI_{\mathrm{e_i}}$), which is defined as the expected cost of performing no inspection subtracted by the expected utility of the inspection, without the inspection cost $C_{\mathrm{e_i}}$:

$$VoI_{\mathrm{e_i}} = E(u, e_0) - (E(u, e_\mathrm{i}) - C_{\mathrm{e_i}}) \tag{34}$$

VoI is calculated for both inspection methods, and is presented in table 11 and compared in fig. 21.

Table 11: VoI for inspection methods e_1 and e_2.

Year	$\boldsymbol{VoI_{e_1}}$ **[NOK]**	$\boldsymbol{VoI_{e_2}}$ **[NOK]**
5	0	0
10	0	2,240
15	2,232	5,980
20	4,178	7,476
25	3,382	6,003
30	833	3,004
35	0	1,504

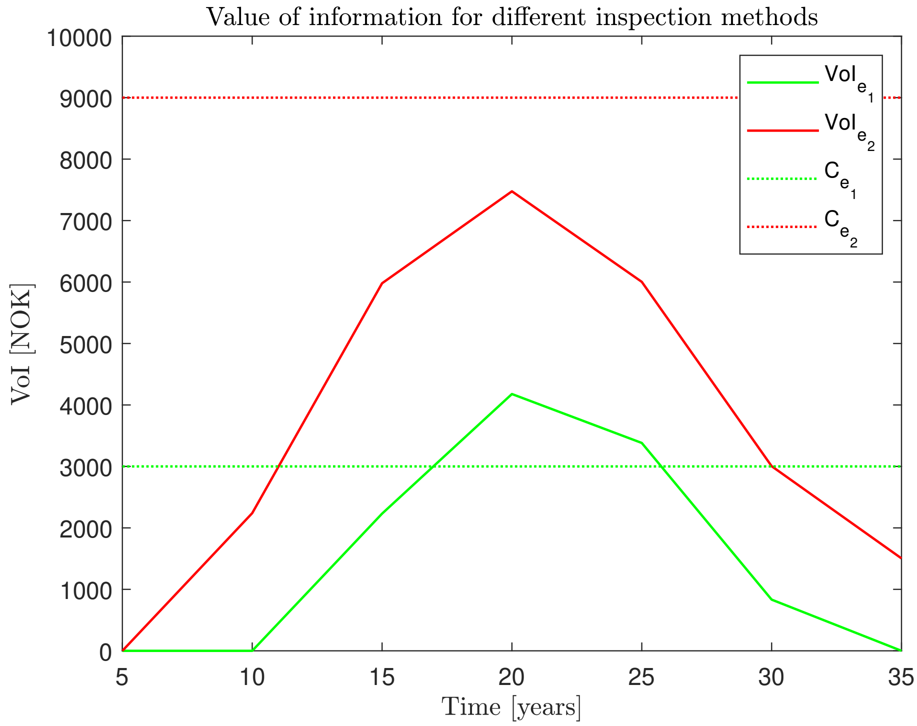

Figure 21: Value of information for inspection methods e_1 and e_2. The values of VoI are given in NOK.

Figure 21 shows that the VoI for inspection method e_1 is equal to zero at years 5, 10, and 35. It can thus be said that little or zero value is gained if inspection e_1 is performed early or very late in the structure's lifetime. Performing inspection method e_1 is only beneficial at years 20-25, since the cost of inspection is lower than the corresponding VoI. For inspection method e_2, the VoI is equal to zero at year 5. During the rest of the structure's lifetime, the VoI is lower than the cost of inspection. That means no benefit is gained from performing inspection method e_2 during those years. Thus, the accuracy of inspection method e_2, which leads to lower expected cost, does not compensate for the high cost of the method.

In other words, what can be seen from fig. 21, is that when VoI_{e_1} is above the line that represents the cost of e_1, it is beneficial. In the case of sensor inspection e_2, VoI_{e_2} is never higher than the cost of inspection e_2.

Discussion

The expected values are dependent on a number of factors, such as the probability of detection and cost of inspection methods. By assuming that the probability of detection of inspection method e_1 is higher, for instance the same as for inspection method e_2, it becomes economical earlier than with previous probability of detection, as shown in fig. 22.

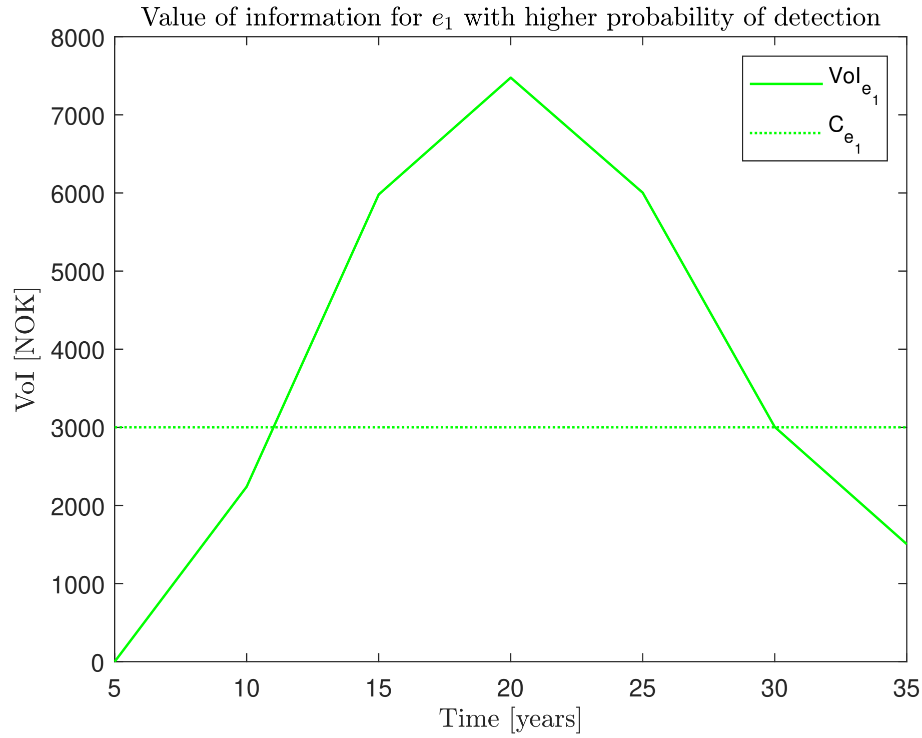

Figure 22: VoI for inspection method e_1, when probability of detection of e_1 is assumed to be higher.

It can be argued that the sensor inspection method e_2 is cheaper than the visual inspection e_1. By lowering the cost of e_2, say $C_{e_2} = 2{,}500$ NOK, inspection method e_2 becomes economical after 10 years and stays that way until year 35. VoI for inspection method e_2, when the cost of inspection is lowered is shown in fig. 23.

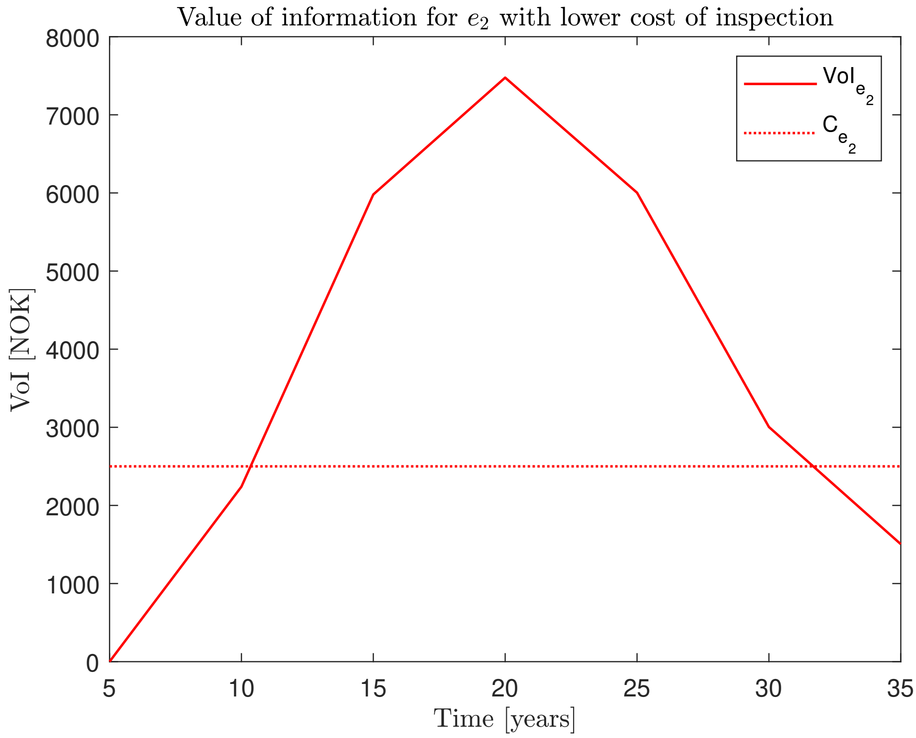

Figure 23: VoI for inspection method e_2, when the cost of inspection is lowered.

3.9 Summary

Decision analysis is an approach that can be used to determine the optimal decision under uncertainty. In priori analysis, the decision maker assigns the prior probabilities of state $P'(\theta)$ and his preferences among possible action-state pairs, expressed by the utility $u(a, \theta)$. The decisions are then based on maximum expected utility criterion. A posterior analysis is based on posterior probabilities $P''(\theta)$ which are found using Bayes' theorem, given the outcome of an experiment $Z = z$. The additional information is expressed in form of a likelihood $P(z \mid \theta)$. In addition, decision analysis can also include the choosing between seveeral possible experiments or inspections prior to choosing the action. This type of analysis, preposterior analysis, involves making a terminal analysis for each possible type of experiment e and the possible corresponding outcome z, which are combined with prior probabilities of the possible outcomes to obtain the expected utility of each experiment.

4 Bridge Inspection Practices by Norwegian Public Roads Administration

4.1 General

The Norwegian Public Roads Administration (NPRA), is responsible for planning, constructing, operating, and maintaining Norway's road and transportation network, including the country's bridges (NPRA n.d. *c*). Bridge management should ensure that society's requirements for a good and uniform standard for safety and functionality of load-carrying structures are met (NPRA n.d. *b*).

The guideline manual V441 - Bridge inspection, published by NPRA (2019), provides guidance and recommendations for inspections of bridges in the road network in Norway. This section presents the main components of the guidelines described in V441.

4.2 Bridge Inspection

Bridge inspection is a part of bridge management and is used to find out whether the bridge still has the load-carrying capacity, traffic safety, durability, and aesthetics for which it was designed and built for originally. The road owner is responsible for the planning and follow up of a bridge inspection. The purpose of these processes is to ensure that the bridges are inspected in accordance with current regulations, that sufficient resources are set aside to carry out the inspections, and that the inspection results and alerts of serious damages are followed up. The bridge inspector is responsible for the preparation and execution of the bridge inspection. The motive is to ensure that the bridge inspections are prepared in accordance with requirements and regulations, the inspection is performed safely, and is registered in the bridge management system Brutus according to this manual, and that the need for possible measures is made visible (NPRA 2019).

4.2.1 Planning and Preparation of a Bridge Inspection

Before a bridge inspection, the inspector must obtain previous inspection documents and prepare for the execution of the inspection. The inspector must review the previous inspection reports, assess the need to involve stakeholders such as landowners, ensure that access to the bridge and necessary inspection equipment are available, carry out risk assessment and job safety analysis, assess the need for securing the workplace, and regulating traffic safely and efficiently at the time of inspection.

4.2.2 Conducting a Bridge Inspection

When carrying out a bridge inspection, damage is inspected, assessed, and registered as described in the manual. Damage assessment, including the degree, extent of damage, and damage consequence, together with description and relevant images, are registered in Brutus. Possible vulnerabilities to the bridge (described in chapter 4.4) are also registered. Damages assessed with degree of consequence 4, must be reported immediately according to an alert plan. After the inspection, the inspector evaluates the need for measures, which can be registered in Brutus.

The manual provides a checklist, which can be used as a tool when performing the inspection. How thorough the inspector should be going through the list, depends on the type of inspection. The checklist consists of recommendations of inspections. It is divided into different categories, depending on what part of the bridge is being inspected. Table 12 shows an example of categories and corresponding recommendations of inspection.

Table 12: Example of category types and corresponding recommendations of inspection (NPRA 2019).

Category	**Recommendation**
Foundation	- Erosion of landfill around abutments. - Damage to piles above water. - Damage to piles under water.
Elements of concrete	- Deformation of structure or elements. - Settlement or movement of abutments, pillars, etc. - Weathering due to frost or other environmental loads. - Visible reinforcement corrosion.
Elements of steel	- Deformation of structure or elements. - Loose or missing screws/nails. - Cracks or fractures in material.
Joints	- Mechanical damages, cracks, fractures.
Cables	- Damage to suspension. - Insulation material is intact.
Railings	- Deformation damage. - Cracks or fractures. - Satisfactory height of railings.

4.2.3 Follow Up of a Bridge Inspection

When the inspection season has started, it must be ensured that this year's planned bridge inspections are on schedule, the selected bridges are inspected, any serious damage consequences are handled, and that the bridge manager is informed of the situation. A control group reviews all damage registrations that have damage consequence 3 or 4. If the group disagrees with the registration, it can be changed in Brutus. In addition, random checks can be made for registered vulnerabilities and complex bridges such as suspension bridges. When choosing which inspections should be checked, one should consider the inspector's experience and competence. All warnings of serious damage consequences must be followed up. The damage is verified and necessary measures are assessed and executed. These measures can for example include reducing the load-carrying capacity, more frequent inspections, and material testing.

4.2.4 Analysing the Results of Inspection and Inspection Intervals

Analyses and assessments of the inspection history, condition, and risk can be performed in order to gain better understanding of the condition and the development of the condition of individual bridges. They can include the analysis of inspection history to identify trends or conditions that need to be corrected, prevented or maintained, analysis of damage development and its rate, and make necessary calculations to support decisions related to further management and maintenance. For next year's inspection season, one can consider risk and vulnerability (ROS) analysis to change the inspection intervals.

4.2.5 Inspection Types

NPRA considers three types of routine inspections, which are carried out throughout the lifetime of the bridge. They are general inspection, simple inspection, and main inspection. The difference of the inspections is the thoroughness and frequency of the inspections performed. If more thorough investigation is needed, or in case of an extraordinary event, special inspection can be performed.

General Inspection is carried out in accordance with manual R610. This inspection includes routine inspection to monitor the function of the road network and is performed by the operating contractor. General inspection is not described further in manual V441. According to manual R610, NPRA (2012), a general inspection should be done every week for national roads and every other week for other roads.

Simple Inspection is performed to detect visible damages that in the short term affects or can affect the load-carrying capacity of the bridge, traffic safety, maintenance, or the environment/aesthetics. Only damages and other conditions that need to be fixed before the next single or main inspection are registered. Simple inspection includes a visual inspection of all elements over water without using access equipment. For larger bridges, it might be necessary to use binoculars to look more closely at details. Foundations are checked without using a diver. Usually, no measurements or material tests are carried out in this type of investigation (NPRA 2019). Simple inspection should be carried out every year, but can be skipped when a main inspection is executed (NPRA 2012).

Main Inspection includes a close visual inspection of the entire bridge. All elements are checked to see if they fulfil their function. When observing larger areas of an element, where expected damage can be detected with certainty from a distance, an area on that element can be chosen to implement a close visual inspection. If necessary, measurements and material testing can be performed in addition. When main inspection is performed, any need for surveys are described with an associated cost estimate for the measures (NPRA 2019). Main inspection should be performed every 5 years according to manual R610, NPRA (2012).

Special Inspection is carried out in the case of an extraordinary event or when a more thorough inspection of a damage is required. This type of investigation can be performed in order to provide a basis for describing and scheduling measures where complicated and/or costly measures are expected. It is carried out on the entire bridge or on individual elements and includes close visual inspection and necessary measurements and material testing. It can also include static calculations to check the capacity of the bridge. The special inspection is performed thoroughly and comprehensively enough so that the type of damage, the degree and extent of damage, the cause of damage, and the consequence of it can be determined. The inspection can include a description of alternative measures with associated cost and lifetime assessments, as well as form the basis for the selection of measures and the preparation of a tender basis (NPRA 2019). Special inspections are not required at specific intervals but are performed when necessary (NPRA 2012).

4.2.6 Surveys and Material Testing

To get a better basis for determining the type of damage, the degree and extent of damage, the cause of damage, and the consequence of damage, or reveal hidden damage, measurements and material testing may be necessary in addition to the visual inspection. The scope of these measures and material testing depend on different factors such as type of inspection, bridge and material type, climate impact, visual observations and management strategies (for example if the bridge should be replaced) of the bridge. It is important to evaluate the need and usefulness of the measurements and testing. That is because they bring additional cost and may affect the appearance of the bridge. When drilling cores for testing, any constructive and durability deterioration of the tested element must be taken into account (NPRA 2019).

4.2.7 Brutus

Brutus is a bridge management system developed and currently used by NPRA to collect condition data about the bridges (NPRA n.d.*b*). Brutus is a information and planning tool for the management, operation, and maintenance of bridges and other structures in the road network. It consists of a database where relevant data from planning, engineering, construction, operation, and maintenance of the structures are registered and stored (NPRA 2009). The database includes information about all bridges on the national and county road network in Norway. Brutus is also used to follow up that road owners inspect the bridges as required by the regulations (NPRA n.d.*b*).

4.3 Damage Assessment

Chapter 3 in manual V441 (NPRA 2019) deals with the damage assessment of bridges. The damage assessment includes the determination of the type of damage, the degree and extent of damage, the cause of damage, and the consequence of it.

Usually, visual inspecton, measurements, and material testing are used as basis for assessing damage. In specific cases it can be necessary to include static calculations and/or structural monitoring over a period of time to obtain an acceptable basis for assessing damage correctly. In order to have a consistent way to evaluate damages, NPRA has developed a classification of the damage type, cause of damage, degree of damage, and the consequence.

4.3.1 Location of Damage

Bridges are divided into bridge types that reflect the superstructure's main load-carrying system and appearance, for example slab bridge or beam bridge. The bridge types are then divided into elements such as abutments and pillars. During the inspection, a damage description is linked to an individual element. The description should include the location of a damage on the bridge and where measurements and material testing have been performed.

4.3.2 Description of Damage and Damage Types

Damage description includes the information about what the damage is, where it is located, and its extent. The extent of damage is quantified, for example by describing the length or volume of a damage. Damages are documented with photos which can also be used to assess the development of the damage by subsequent inspections. The damage description can also include possible consequences of the damage to the entire bridge. The damage types that are described in the manual and are registered in Brutus are:

- Damage not related to material;
- Damage of ground;
- Damage of concrete;
- Damage of steel;
- Damage of stone;
- Damage of timber;
- Damage of surface course/moisture insulation;
- Deficiency;

- Other damage/deficiency.

The types listed above are then divided into subgroups describing different types of damages. If the same type of damage is identified at several places in the same element, the type of damage is registered once, but with the highest degree of damage.

Chapter 6 in the manual provides a general description of the different types of damages. Each damage type is described with the following information:

Description of the type of damage. Includes information about what the damage is, where it is located, and its extent;

Degree of Damage for some types of damages there exists a scale for the degree of damage. For other damages, the sample collection in appendix V1 of manual V441 can be used to determine the degree of damage;

Damage Consequence as described in chapter 4.3.4;

Cause of Damage the highest level of cause of damage is described;

Suggestion for Measures to follow up and repair damages;

Measurements/Material Testing that can be performed to uncover damage that is not visible, ascertain the extent, and assume future development and/or cause of damage.

4.3.3 Degree of Damage

The degree of damage is used to indicate the seriousness of the damage. NPRA has defined 4 levels of damages, which are listed below. The reference level is undamaged element.

1 Small damage;

2 Medium damage;

3 Large damage;

4 Extreme damage.

The degree of damage is based on the observed condition, and any measurements and testing done on the element being inspected. For some types of damages, there is a scale for the degree of damage, which is described in chapter 6 of the manual.

4.3.4 Damage Consequence

Damage consequence is described by the consequence type and the degree of consequence.

Consequence Type indicates the consequence a damage has for the bridge, its users and the surroundings. NPRA defines the following consequence types:

- **B** Damage than can affect the load-carrying capacity;
- **T** Damage that can affect road safety;
- **V** Damage than can increase maintenance cost;
- **M** Damage that can affect the environment/aesthetics.

The list above is in the order of most serious to least serious consequence type, with B being the most serious type.

The **Degree of Consequence** indicates how serious the damage is to the entire bridge, its users and the surroundings. NPRA defines the following degrees of consequence:

- **1** Small consequence. No need for action;
- **2** Medium consequence. Whether measures should be registered in Brutus should be considered;
- **3** Considerable consequence. Proposals for measures are registered in Brutus. Inspection intervals must be considered;
- **4** Great consequence. The bridge manager/client is contacted immediately.

Damage Consequence for Load-Carrying Capacity

Damages that affect the load-carrying capacity can be for example settlements or deformation of abutments which can affect the beam or slabs, corrosion, and cross sectional reduction on steel beams, and mechanical damage due to fracture in concrete beams. A damage with a great consequence on the load-carrying capacity would therefore be a 4B damage consequence.

Damage Consequence for Traffic Safety

Damages that affect traffic safety can, for example, be railings with a damage where they are fixed to the ground, cross sectional reduction or deformations, or a fallout such as loose concrete from a structure. A 1T damage consequence is a damage that has small consequence for traffic safety.

Damage Consequence for Maintenance Costs

When determining consequence, account is taken of how quickly a damage develops and how large the cost increase may be if measures are not executed. Damages that affect maintenance costs can be, for example, steel surfaces with degraded surface, and reinforcement corrosion. A damage with medium consequence for maintenance costs would have the damage consequence 2V.

Damage Consequences for the Environment/Aesthetics

Damages that can affect the environment/aesthetics can for example be graffiti, and deficiencies in joint constructions (which leads to noise). A 3M damage consequence would have considerable impact on the environment/aesthetics.

4.3.5 Assessment of Damages

Damages can often lead to secondary damages. To be aware of the connection is important and primary damages should be treated before the secondary damages, as repairs of only secondary damages are rarely successful. An example of a primary damage and possible secondary damage is a small or damaged cover, which can lead to reinforcement corrosion.

Critical damages that have already reduced the load-carrying capacity or traffic safety are easy to decide on since measures must be executed quickly. It is however more difficult to assess damages that are under development. In those cases it is important to determine how long the damage has been developing and how fast it will develop in the future, in addition to reviewing documents from previous inspections. Sometimes damages can be initiated without visible damage, for example when chloride penetrates in to the concrete without having reached the reinforcement yet, thus reinforcement corrosion has not yet occurred. When assessing the rate of degradation, it is important to note that damage and damage development can be different on different parts of the bridge/bridge element.

The manual provides an illustration of the damage development processes, shown in figure 24, and defines the four development processes as:

No Damage Development either the damage is very small or so serious that it needs to be fixed immediately. Could be for example due to a collision. Can lead to secondary damages if no measures are taken;

Reducing Damage Development deformation of the soil is a type of damage where the damage development often decreases over time;

Linear Damage Development road surface wear often has a linear damage development;

Accelerating Damage Development example of a damage with accelerating development is when chloride penetrates into a concrete and reinforcement corrosion.

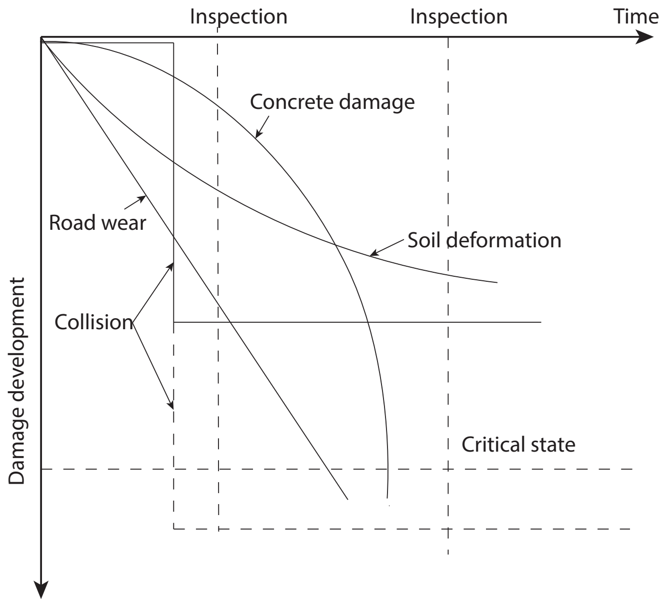

Figure 24: Examples of damage development (NPRA 2019).

4.3.6 Cause of Damage

In order to have a successful maintenance measure, it is necessary to identify the cause of damage. The basis for determining the cause of damage is visual inspection and possibly measurements and material testing in addition. To have a good knowledge of the design, construction, and management of bridges is also important.

NPRA divides the main causes of damage into nine categories which are then divided into subcategories. They are registered in Brutus. The main causes are listed in the manual and are as follows:

- 10 Design error;
- 20 Material defect;
- 30 Execution Errors;
- 40 Lack of operation/maintenance;
- 50 Environmental attacks;
- 60 Loading;
- 70 Accidental loading;
- 80 Damage from usage;
- 90 Other/unknown.

4.3.7 Priority Scheme

In order to prioritise actions in the evaluation of damages by NPRA, a priority scheme has been developed. The priority of action for the damage is suggested to be the product between damage degree and the consequence degree. From this, a priority matrix can be developed. The higher the product of a damage, the higher the priority of the damage. The priority matrix is shown in figure 25. Green indicates low priority, yellow indicates medium priority and red indicates high priority in the damage assessment (Solheim 2018).

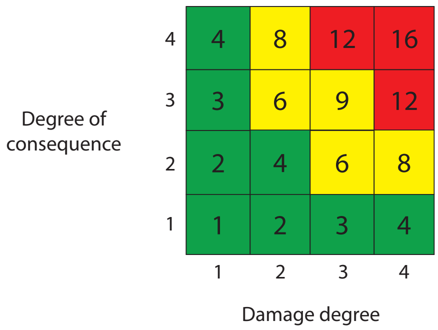

Figure 25: Priority matrix (Solheim 2018).

4.4 Vulnerability Assessment

A vulnerability for a bridge is when a property of the bridge or its surroundings can affect its functionality and safety, but is not considered a damage. These vulnerabilities are registered in Brutus. The following vulnerabilities are listed in manual V441 (NPRA 2019):

- Fire and explosion;
- Flooding;
- Landslides;
- Ice passage;
- Collision;
- Traffic flow;
- Traffic safety;
- Load-carrying capacity.

4.5 Measures

Based on the inspection and the condition of the bridge, measures can be suggested. Chapter 5 in the manual gives a description of the different types of measures that can be registered in Brutus. The following measures are described:

Operational Measures planned tasks that are necessary for the bridge to function as intended. May include cleaning of bridge elements and renovation;

Maintenance measures to repair critical damages or damages that cannot be planned long term, planned measures to maintain the standards of the bridges, and planned measures to restore the functionality of a damaged element;

Renewal The remaining life of the existing bridge is taken out and the construction of a new bridge is considered;

Reinforcement/Strengthening measures that increase the load-carrying capacity of a bridge or an element, damaged or not damaged, relative to the original load-carrying capacity;

Rebuilding/Modification measures that change the function, land use, or standard of a bridge or a bridge element, in order to improve the accessibility and/or traffic safety. Can also be an alternative to perform maintenance measures where there is great damage;

Load-Carrying Capacity Check is done when a damage, defect, or deficiencies have been identified that may affect the load-carrying capacity;

ROS-Analysis can be carried out when a damage, design, or surroundings make the bridge exposed or vulnerable to events;

Measurements/Surveying used in addition to visual control when there is need for additional follow-up and/or reveal hidden damages;

Material Testing used in addition to visual control when there is need for additional follow-up and/or reveal hidden damages.

4.6 Risk and Vulnerability Analysis

Risk and vulnerability (ROS) analysis involves a systematic process to assess and document the condition of the bridge, map possible undesirable events and vulnerabilities, and estimate the probability, consequence of, and risk of failure (Dahle 2019). NPRA does not provide official guidelines for ROS-analyses.

4.7 Summary

When conducting a bridge inspection, the inspectors are looking for any defects, damages, or potential problems that may require maintenance. NPRA defines and conducts three types of routine inspections, which all have different intervals between the inspections: general inspection, simple inspection, and main inspection. In addition, a special inspection can be carried out when necessary. NPRA uses the bridge management system Brutus to register data collected from the inspections and to assess the need for measures. The manual V441 (NPRA 2019) provides guidelines on how to document and assess the damages.

NPRA has developed a classification of the damage type, degree of damage, cause of damage, and the consequence, which is used to evaluate and prioritise the damages in a consistent way. The damage types are described in manual V441. They should be registered in Brutus. Finding the cause of damage is important in order to have a successful maintenance measure. The cause of damage is also registered in Brutus. The degree of damage indicates the seriousness of the damage. Damage consequence consists of the consequence type and the degree of consequence. The consequence type indicates the consequence a damage has for the bridge and the degree of consequence how serious the damage is to the entire bridge. NPRA uses a priority matrix to prioritise actions. Based on knowledge from the inspection and other information that can have impact on the decision, the bridge owner can decide what action is required and when it is required.

5 Interview About Inspection and Maintenance Processes at NPRA

5.1 Introduction

In order to get further insight into the inspection practises at NPRA, an interview with an employee at the administration was carried out.

5.2 Methods

An interview was conducted with Tor Anders Hagstrøm, a senior engineer in the Bridge and Ferry Quay department at NPRA. The interview was performed on April 6th 2022. The objective of the interview was to investigate the decision processes for bridges regarding inspection and maintenance. The script of questions can be found in appendix C.

5.3 Findings

5.3.1 Inspection Intervals

The planned inspections of bridges are always carried out in accordance with requirements and regulations. According to Hagstrøm (2022), they have necessary resources (funds, people, etc.) to carry out all inspections. The only exception is where unexpected incidents happen. In those cases, time will still be found to perform the inspection. The inspections are scheduled ahead of time on a settled date. The inspection dates can however be changed based on evaluations.

In 2019, ROS-analysis was introduced as a tool to optimise the number of required inspections. The inspection intervals can be changed if a ROS-analysis shows that it is justifiable. The ROS-analysis is used to eliminate the simple inspections, which as mentioned before, are performed annually. The frequency of the main inspections can also be extended or reduced, but that is not a standard procedure at NPRA. Performing ROS-analysis is currently not a requirement at NPRA when inspecting a bridge, but the inspectors are allowed to use it as a tool when they feel it is necessary.

After the inspection, the inspector is the one that knows the most about the condition of the bridge, thus performing a ROS-analysis at that time would be optimal, as it would be based on a recent status of the structure. Depending on the condition of the bridge and the findings the inspector has discovered, the intervals can be changed. Other factors that are taken into consideration in the ROS-analysis is the bridge material, the age of the bridge, and what design rules were governing at the time of design. For different bridges, there may be different approaches to do the

ROS-analysis.

The planned inspections are usually not put on hold. If a problem comes up that results in the inspection not being performed, an emphasis is put on carrying out the inspection within the same inspection year. If that is not possible, the bridge can be evaluated from data the inspector has, and the inspection can be moved to the following year, preferably early. However, those cases are usually an exception.

The ROS-analysis applies for existing bridges, newly constructed bridges, and bridges that have recently been repaired. Newly constructed and recently repaired bridges therefore follow the same inspection intervals as existing bridges, and also have the possibility of being extended or delayed, if a ROS-analysis is carried out. After a construction of a new bridge, a 'handover' inspection is performed by employees of NPRA together with the contractor, to see if there is something they are not satisfied with before the contractor leaves the site. Hagstrøm (2022) says that it is good practise to inspect the bridge more closely in the simple inspection while the bridge is still guaranteed (the guarantee period could for example be 3 years), so that NPRA can claim a repair without being the ones that have to pay for it.

In order for a tool such as ROS-analysis to function, it is important to have a good system for handling the data from the analysis. That is something that NPRA is working on, according to Hagstrøm (2022). He visions that you would also be able to see the ROS-status of the bridge in the Brutus database.

5.3.2 Analysing Inspection Results

After an inspection, the inspector generates a report with the findings. He can report the inspection findings immediately, but at the office, he has access to more information regarding the bridge condition and history. He can perform calculations and discuss with the experts that perform the capacity assessments of the bridge. By doing that, the report is improved enormously. A large part of the findings is something that has to be reported and followed up, but has no consequences. Thus, there is not a reason to rush the report. If critical damages are found during the inspection, they are reported immediately. In the worst case, the bridge has to be closed. Other cases could for example be that the bridge needs to be repaired within the next three months. Thus, the registration of inspection results and timing of maintenance are not always registered right after an inspection has been carried out. Hagstrøm (2022) mentions that is is very import to not start stressing unnecessarily because that way you start to focus on the wrong things at the wrong time.

5.3.3 Maintenance Work after Inspection

In addition to registering the inspection results in Brutus, the inspector should always report if there is some physical maintenance required. It might for example be the statement that a specific damage needs to be fixed within a time period, for example 6 months or 2 years. A large part of damages do not need urgent repair.

They are still registered in the system in order for bridge owners to have a good overview over the bridge condition. At some point in time, they may have a list of repairs that need to be done and will start a project to repair all damages at once. The bridge owners are the ones that make a decision to perform maintenance measures. However, NPRA can make suggestions based on their results from inspections, and can put force on the bridge owners if they feel like a damage should be repaired immediately. They usually listen to their advice (Hagstrøm 2022).

NPRA does not carry out preventive maintenance, that is, regular and routine maintenance of the bridge's elements. Instead, maintenance measures are only done if an inspection justifies the need for it.

5.3.4 Surveys and Material Testing

Material testing and/or measurements may be necessary in addition to inspections. NPRA performs this in special cases, for example when some special transport is coming and NPRA is struggling to prove theoretically that the bridge has sufficient capacity. Different methods can be used to further investigate a damage. The choosing of methods depends on the condition of each individual bridge and on the experts and their experience. Material testing needs to be very well reasoned and planned. Material experts are involved along with bridge analysis experts. The bridge analysis experts would decide where on the bridge the samples should be taken from, as it is not desired to destroy the most critical points in the bridge.

Hagstrøm (2022) is not aware that there are any rules regarding there being a rational on how many measurements should be done in order for them give results that a decision can be based on. According to him, people that are doing the bridge assessment should be involved in the process, so the requirements and history of the bridge can be discussed.

5.3.5 Safety and Reliability Assessments

The assessment of the structural safety of bridges is carried out according to the Eurocode standards. In a safety assessment of an existing bridge, there are a lot of gray zones, since regulations have changed (traffic loads increased, bridges not designed for earthquake loads, etc.). Hagstrøm (2022) says NPRA are doing their best to ensure the safety of bridges at each time, by carrying out inspections and follow classifications and rules. But one can never be fully certain that a bridge will not collapse. For one, human errors do happen. What makes an inspection better? Using a drone could be useful, but the drone is just a tool which helps to perform a better inspection, not necessarily what makes the inspection good. The most important part is the inspector and the evaluation he does after an inspection.

5.3.6 Future Condition of Norwegian Bridges

Hagstrøm (2022) is not concerned about bridge condition in Norway now or in the coming years. Compared to bridges in the Unites States for example, Norwegian bridges are in good condition in his opinion. However, looking at bridge collapses around the world gives a good reminder to look after our bridges. He thinks that big focus should be put on inspecting bridges, since it is a large part of ensuring their safety. The inspections cannot be neglected.

Hagstrøm (2022) believes that there are enough funds to do maintenance on critical damages, since in those cases, money is of no concern. He says that it is difficult to predict the future, but with the ongoing daily work at NPRA, year by year, with focus on maintenance, there will not be a problem. He is more concerned about unexpected natural events that can not be controlled. Finally, he mentioned that money should be put into maintenance before we start to get worried. It is better to stop the development of damages before they get into critical situations.

5.4 Summary

Bridge inspections are performed as scheduled at NPRA. This applies for existing, newly constructed, and newly repaired bridges. Recently, ROS-analysis was introduced, which can be used to optimise the number of required inspections. Ideally, a ROS-analysis of a bridge would be visible in the Brutus database.

A report is made with the findings of an inspection. To get a better basis for the planning and execution of follow-up maintenance, data regarding the bridge previous condition and history can be used. Most findings do not require immediate actions, thus the report does not need to be rushed. However, critical damages are reported immediately. NPRA can make suggestions for maintenance work which are ultimately decided by the bridge owners. NPRA does not perform predictive maintenance of bridge elements but instead maintenance is done if an inspection results in it being recommended.

Material testing and/or measurements are carried out in special cases. It involves material- and bridge analysis experts. To what extent the measurements should be done is decided by the experts in each case.

The assessment of the structural safety of bridges is carried out according to Eurocode standards. For existing bridges there exists a lot of uncertainty in addition to regulations and requirements changing with time, thus evaluations tend to be conservative.

6 Service Life Modelling

6.1 Basic Service Life Concepts

6.1.1 Design for Durability

The understanding of the importance of durability has greatly increased over the last 40 years. Durability was not considered in design codes previously, but in the 1980s, the term 'durability' started being considered to be equally important as the term 'structural safety' in national design codes. In Europe, the current method to design for durability is based on deemed to satisfy rules, where durability is assumed to be achieved by specifying limiting values for concrete composition according to EN 206 and construction according to EN 13670. In addition, EN 1992 provides the requirements for specific durability-related properties with regard to minimum cover and maximum crack widths (Geiker et al. 2021).

Another way to design for durability is the performance-based approach, where the owner sets the requirements with regards to service life and required documentation. The contractor then proposes various solutions to fulfil the requirements. To make it possible to compare and validate the proposed solutions, service life models, reliable performance test methods, and acceptance criteria are necessary (Geiker et al. 2021). Several service life models have been developed, such as DuraCrete and *fib* Model Code. To a large extent, the DuraCrete guidelines formed the basis for the *fib* Model Code for service life design, published in 2006 (*fib* Bulletin 34 2006).

6.1.2 *fib* Model Code Design

The *fib* Bulletin 34 (2006) Model Code contains several approaches to service life design. In principle, they avoid or minimise deterioration caused by environmental actions. The design approaches are categorised as full probabilistic, partial factor, deemed to satisfy, and avoidance of deterioration. Any of the methods can be used for service life design, although the full probabilistic approach should only be used for exceptional structures. These methods can be applied for the design of new structures and for the assessment of remaining service life of existing structures. Figure 26 illustrates these design approaches.

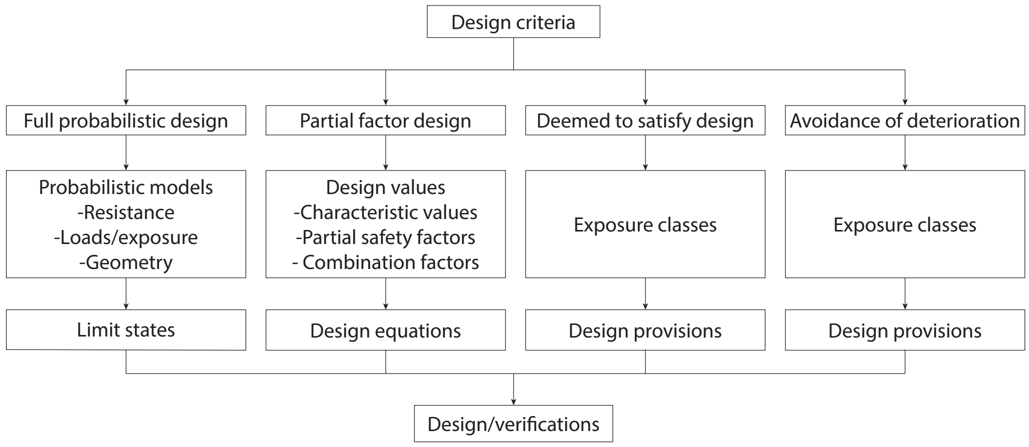

Figure 26: A flow chart of design approaches described in fib model code. Based on *fib* Bulletin 34 (2006).

The deterioration mechanism in question is quantified with models that describe the process physically and/or chemically with sufficient accuracy. The environmental actions are also modelled, with statistically quantified environmental parameters (*fib* Bulletin 34 2006). The selection of acceptance criteria, i.e., the limit states and the probabilities of exceeding them, is an important part of durability design (Geiker et al. 2021). Appendix B provides an overview of structural reliability analysis, including an introduction of limit states. Different limit states may be considered for structures affected by deterioration, including:

- Depassivation of reinforcement caused by carbonation;
- Cracking due to reinforcement corrosion;
- Spalling of concrete cover due to reinforcement corrosion;
- Collapse due to loss of cross section of the reinforcement.

The calculation of the probability that the limit states are exceeded is done by applying the models described earlier. Broad accepted models exist for the verification of a limit state. It is commonly accepted that the safety of structures is expressed in terms of reliability (reliability index β) (*fib* Bulletin 34 2006).

6.2 Service Life Design Verification, Full Probabilistic Method

Various deterioration mechanisms are treated in *fib* Bulletin 34 (2006), including carbonation induced corrosion and chloride induced corrosion. In the remainder of this chapter, an overview of full probabilistic models for these deterioration mechanisms will be presented.

6.2.1 Service Life Models, Carbonation Induced Corrosion

A full probabilistic design approach for the modelling of carbonation induced corrosion of uncracked concrete has been developed. The approach is based on the limit state where the concrete cover a is compared to the carbonation depth x_c at time t.

Propagation of Carbonation

In *fib* Bulletin 34 (2006), the propagation of the carbonation front from the concrete surface is described as:

$$x_{\mathrm{c}}(t) = \sqrt{2 \cdot k_{\mathrm{e}} \cdot k_{\mathrm{c}} \cdot R_{\mathrm{NAC,0}}^{-1} \cdot C_{\mathrm{s}}} \cdot \sqrt{t} \cdot W(t) \tag{35}$$

where

$x_{\mathrm{c}}(t)$: carbonation depth at time t [mm];

t: time [years];

k_{e}: environmental function [-];

k_{c}: execution transfer parameter [-];

$R_{\mathrm{NAC,0}}^{-1}$: inverse effective carbonation resistance of concrete (65% RH) determined at a certain point of time t_0 on specimens with the normal carbonation test NAC [(mm^2/years)/(kg/m^3)];

C_{s}: CO_2-concentration [kg/m^3];

$W(t)$: weather function.

Limit State for Carbonation Induced Corrosion

The depassivation limit state for carbonation induced corrosion in uncracked concrete can be formulated according to eq. (36):

$$p_{\mathrm{dep}} = p[a - x_{\mathrm{c}}(t_{\mathrm{SL}}) < 0] < p_0 \tag{36}$$

where

p_{dep}: probability that depassivation occurs;

a: concrete cover [mm];

$x_{\mathrm{c}}(t_{\mathrm{SL}})$: carbonation depth at time t_{SL} [mm];

t_{SL}: design service life [years];

p_0: target failure probability.

Parameter Quantification

Concrete Cover a

The concrete cover a is chosen during design, but can however vary after construction. For large concrete covers, normal distribution is commonly used. For smaller concrete covers, distributions such as lognormal, beta-, weibull(min)-, or the neville distribution should be chosen.

$$a \sim \mathcal{N}(\mu_{\mathrm{a}}, \sigma_{\mathrm{a}}) \tag{37}$$

Design Service Life t_{SL}

The design service life is denoted by t_{SL}. Table 13 shows examples of types of structures and their indicative design service life values.

Table 13: Indicative values for the design service life t_{SL}. Based on *fib* Bulletin 34 (2006).

Design service life t_{SL} [years]	**Examples**
10	Temporary structures
10-25	Replaceable structural parts, e.g. gantry girders, bearings
15-30	Agricultural and similar structures
50	Building structures and other common structures
100	Monumental buildings structures, bridges, and other civil engineering structures

Environmental Function k_e

The environmental parameter k_{e} considers the carbonation resistance of the concrete by taking into account the influence of the humidity level on the diffusion coefficient. The environmental function can be described as:

$$k_{\mathrm{e}} = \left(\frac{1 - \left(\frac{RH_{\mathrm{real}}}{100}\right)^{f_{\mathrm{e}}}}{1 - \left(\frac{RH_{\mathrm{ref}}}{100}\right)^{f_{\mathrm{e}}}} \right)^{g_{\mathrm{e}}} \tag{38}$$

where

RH_{real}: relative humidity of the carbonated layer [%];

RH_{ref}: reference relative humidity [%], constant parameter with value 65;

g_{e}: exponent [-], constant parameter, value: 2.5;

f_{e}: exponent [-], constant parameter, value: 5.0.

A right-skewed distribution is commonly used to describe the relative humidity, RH_{real}, in European climate conditions. The reference relative humidity, RH_{ref}, is chosen in accordance with the test conditions for determining the carbonation resistance ($R^{-1}_{\mathrm{ACC,0}}$) of the concrete.

Execution Transfer Parameter k_c

The execution transfer parameter, k_{c}, takes account of the influence of curing on the effective carbonation resistance. By means of Bayesian regression, the execution transfer parameter can be estimated:

$$k_{\mathrm{c}} = \left(\frac{t_{\mathrm{c}}}{7}\right)^{b_{\mathrm{c}}} \tag{39}$$

where

b_{c}: exponent of regression [-], $b_{\mathrm{c}} \sim \mathcal{N}(\mu_{b_{\mathrm{c}}}, \sigma_{b_{\mathrm{c}}})$;

t_{c}: curing period [d].

Inverse Carbonation Resistances $R^{-1}_{NAC,0}$ and $R^{-1}_{ACC,0}$

The inverse effective carbonation resistance is dependent on the w/c ratio and the type of binder of the concrete, and can be determined using a accelerated carbonation test (ACC-test method). In the ACC-test, the inverse carbonation resistance, $R^{-1}_{\mathrm{ACC,0}}$, is found by testing pre-stored concrete specimens under defined conditions at a reference time t_0 in a laboratory. The ACC-test method is further described in *fib* Bulletin 34 (2006). In order to transform the results gained from the test into the inverse carbonation resistance (determined under natural conditions, NAC) $R^{-1}_{\mathrm{NAC,0}}$, the factors k_{t} and ϵ_{t} are introduced:

$$R^{-1}_{\mathrm{NAC,0}} = k_{\mathrm{t}} \cdot R^{-1}_{\mathrm{ACC,0}} + \epsilon_{\mathrm{t}} \tag{40}$$

where

$R^{-1}_{\mathrm{ACC,0}}$: inverse effective carbonation resistance of dry concrete, determined at a certain point of time t_0 on specimens with the accelerated carbonation test ACC [(mm^2/years)/(kg/m^3)], $R^{-1}_{\mathrm{ACC,0}} \sim \mathcal{N}(\mu_{\mathrm{R}^{-1}_{\mathrm{ACC,0}}}, \sigma_{\mathrm{R}^{-1}_{\mathrm{ACC,0}}})$;

k_{t}: regression parameter which considers the influence of test method on the ACC-test [-], $k_{\mathrm{t}} \sim \mathcal{N}(\mu_{\mathrm{k_t}}, \sigma_{\mathrm{k_t}})$;

ϵ_{t}: error term considering inaccuracies which occur conditionally when using the ACC test method [(mm^2/years)/(kg/m^3)], $\epsilon_{\mathrm{t}} \sim \mathcal{N}(\mu_{\epsilon_{\mathrm{t}}}, \sigma_{\epsilon_{\mathrm{t}}})$.

Environmental Impact C_s

The CO_2 concentration of the ambient air represents the direct impact on the concrete structure, which can be described by:

$$C_{\mathrm{s}} = C_{\mathrm{S,atm}} + C_{\mathrm{S,emi}} \tag{41}$$

where

C_{S}: CO_2 concentration [kg/m^3];

$C_{\mathrm{S,atm}}$: CO_2 concentration of the atmosphere [kg/m^3];

$C_{\mathrm{S,emi}}$: additional CO_2 concentration due to emissions sources [kg/m^3].

For usual structures, C_{s} can be considered to be equal to $C_{\mathrm{S,atm}}$. For structures such as road tunnels, or when combustion engines are used, increased CO_2 concentrations can be applied. The atmospheric concentration, $C_{\mathrm{S,atm}}$, is considered to be normally distributed:

$$C_{\mathrm{S,atm}} \sim \mathcal{N}(\mu_{\mathrm{C_{S,atm}}}, \sigma_{\mathrm{C_{S,atm}}}) \tag{42}$$

Weather Function $W(t)$

The weather function takes into account the meso-climatic conditions due to wetting events of the concrete surface. The function is described by:

$$W = \left(\frac{t_0}{t}\right)^{\frac{(\mathrm{p_{SR}} \cdot \mathrm{ToW})^{b_{\mathrm{w}}}}{2}} \tag{43}$$

where

t_0: time of reference [years], constant parameter with value: 0.0767 (28 days);

ToW: time of wetness [-];

p_{SR}: probability of driving rain [-];

b_{w}: exponent of regression, $b_{\mathrm{w}} \sim \mathcal{N}(\mu_{b_{\mathrm{w}}}, \sigma_{b_{\mathrm{w}}})$.

ToW is the average number of rainy days per year, where a rainy day is defined by a minimum amount of precipitation water of $h_{\mathrm{Nd}} = 2.5$ mm per day. Probability of driving rain, p_{SR}, is the average distribution of the wind direction during rain events.

6.2.2 Service Life Models, Chloride Induced Corrosion

Similar to a carbonation induced corrosion, a full probabilistic design approach for the modelling of chloride induced corrosion of uncracked concrete has been developed.

Modelling Chloride Ingress

The proposed probabilistic model for chloride penetration is based on a limit state equation, in which the critical chloride concentration C_{crit} is compared to the actual chloride concentration at the depth of the reinforcement steel at time t:

$$C_{\mathrm{crit}} = C(x = a, t) = C_0 + (C_{\mathrm{S},\Delta\mathrm{x}} - C_0) \cdot \left[1 - \mathrm{erf}\frac{a - \Delta x}{2 \cdot \sqrt{D_{\mathrm{app,C}} \cdot t}}\right] \tag{44}$$

where

C_{crit}: critical chloride content [wt.-%/c];

$C(x,t)$: content of chlorides in the concrete at a depth x from surface and at time t [wt.-%/c];

C_0: initial chloride content of the concrete [wt.-%/c];

$C_{\mathrm{S},\Delta\mathrm{x}}$: chloride content at a depth Δx and a certain point of time t [wt.-%/c];

x: depth with a corresponding content of chlorides $C(x,t)$ [mm];

a: concrete cover [mm];

Δx: depth of the convection zone (concrete layer, up to which the process of chloride penetration differs from Fick's 2nd law of diffusion) [mm];

$D_{\mathrm{app,C}}$: apparent coefficient of chloride diffusion through concrete [mm^2/years];

t: time [years];

erf: error function.

The model given by eq. (44) is based on Fick's 2nd law of diffusion, where transport of chlorides are assumed to be diffusion controlled. However, in the convection zone, a zone where the surface is often exposed to a frequent change of wetting and subsequent evaporation, the transport mehanisms are not mainly diffusion controlled.

Thus, Fick's second law of diffusion is not satisfactory for eq. (44). In the equation, the data of the convection zone is thus neglected and Fick's 2nd law of diffusion is applied, starting at depth Δx with a substitute surface concentration $C_{\mathrm{S},\Delta\mathrm{x}}$.

The apparent coefficient of chloride diffusion is determined by eq. (45):

$$D_{\mathrm{app,C}} = k_{\mathrm{e}} \cdot D_{\mathrm{RCM,0}} \cdot k_{\mathrm{t}} \cdot A(t) \tag{45}$$

where

k_{e}: environmental transfer variable [-];

$D_{\mathrm{RCM,0}}$: chloride migration coefficient [mm^2/a];

k_{t}: transfer parameter [-];

$A(t)$: subfunction considering the 'ageing' [-]

$$A(t) = \left(\frac{t_0}{t}\right)^{\mathrm{a}} \tag{46}$$

a: ageing exponent [-];

t_0: reference point of time [years], constant parameter with value: 0.0767 (28 days).

Limit State for Chloride Induced Corrosion

The limit state for depassivation due to chloride induced corrosion in uncracked concrete can be formulated according to eq. (47):

$$p_{\mathrm{dep}} = p[C_{\mathrm{crit}} - C(a, t_{\mathrm{SL}}) < 0] < p_o \tag{47}$$

where

p_{dep}: probability that depassivation occurs;

C_{crit}: critical chloride content to achieve depassivation of the reinforcement;

$C(a, t_{\mathrm{SL}})$: chloride content at depth a and time t [wt.-%/binder content];

a: concrete cover [mm];

t_{SL}: design service life [years];

p_0: target failure probability.

Parameter Quantification

Chloride Migration Coefficient $D_{RCM,0}$

The chloride migration coefficient, $D_{\mathrm{RCM,0}}$, is an important parameter for the description of the material properties in the chloride induced corrosion model. $D_{\mathrm{RCM,0}}$ can be assumed to follow normal distribution:

$$D_{\mathrm{RCM,0}} \sim \mathcal{N}(\mu_{\mathrm{D_{RCM,0}}}, \sigma_{\mathrm{D_{RCM,0}}}) \tag{48}$$

$D_{\mathrm{RCM,0}}$ should be quantified according to a rapid chloride migration method, which is referred to in *fib* Bulletin 34 (2006). For the assessment of existing structures, $D_{\mathrm{RCM,0}}$ does not need to be quantified. Instead, D_{app} can be derived directly from chloride profiles taken from the chloride exposed structure at different times.

Transfer Parameter k_t and Ageing Exponent a

The transfer parameter k_{t}, and ageing exponent a are introduced in eq. (45) to take into account that the apparent diffusion coefficient D_{app} is subject to considerable scatter and tends to reduce with increasing exposure time.

Environmental Transfer Variable k_e

The environmental transfer variable, k_{e}, takes into account the influence of T_{real} on the diffusion coefficient.

$$k_{\mathrm{e}} = \exp\left(b_{\mathrm{e}}\left(\frac{1}{T_{\mathrm{ref}}} - \frac{1}{T_{\mathrm{real}}}\right)\right) \tag{49}$$

where

b_{e}: regression variable [K], $b_{\mathrm{e}} \sim \mathcal{N}(\mu_{\mathrm{b_e}}, \sigma_{b_e})$;

T_{ref}: standard test temperature [K], constant parameter, value: 293;

T_{real}: temperature of the structural element or the ambient air [K], $T_{\mathrm{real}} \sim \mathcal{N}(\mu_{\mathrm{T_{real}}}, \sigma_{\mathrm{T_{real}}})$;

Initial Chloride Content of the Concrete

The initial chloride content of the concrete, C_0, takes into account the chloride content that could be caused by chloride contaminated aggregates, cements, or water used for the concrete production. The distribution of C_0 can be assumed to be uniform over the whole cross section.

Chloride Contents $C_{S,\Delta x}$ at Depth Δx

The chloride content at the surface of the concrete, C_s, and the substitute surface content at a depth Δx are directly impacted by the material properties of the concrete, and geometrical and environmental conditions. The material properties are mainly the type of binder and the concrete composition. The environmental impact mainly includes the equivalent chloride concentration of the ambient solution. The geometry of the structure and the distance to the source of chlorides can also an important factor. The information needed in order to determine C_s and $C_{\mathrm{S},\Delta\mathrm{x}}$ are shown in fig. 27.

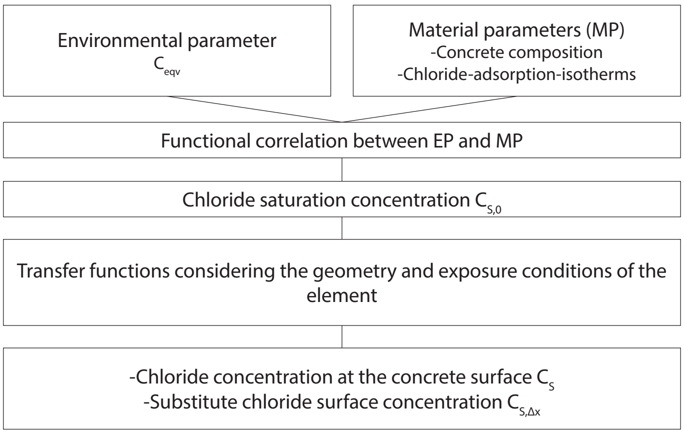

Figure 27: Flowchart of what information is needed in order to determine C_s and $C_{\mathrm{S},\Delta\mathrm{x}}$. Based on *fib* Bulletin 34 (2006).

The potential chloride impact, C_eqv depends on the chloride source. For marine or coastal structures, C_eqv is equal to the natural chloride content of sea water, $C_\mathrm{0,M}$ [g/l]. In the case of chloride contaminated water due to de-icing salt $C_\mathrm{0,R}$, the chloride concentration varies significantly more than sea water with a comparable natural chloride content, making it difficult to quantify. *fib* Bulletin 34 (2006) provides eq. (50), which can be used to describe $C_\mathrm{0,R}$:

$$C_\mathrm{eqv} = C_\mathrm{0,R} = \frac{n \cdot c_\mathrm{R,i}}{h_\mathrm{S,i}} \tag{50}$$

where

$C_\mathrm{0,R}$: average chloride content of the chloride contaminated water [g/l];

n: average number of salting events per year [-];

$c_\mathrm{R,i}$: average amount of chloride spread within one spreading event [g/m^2];

$h_\mathrm{S,i}$: amount of water from rain and melted snow per spreading period [l/m^2].

Material parameters that are needed in order to calculate the chloride saturation content $C_\mathrm{S,0}$, are the concrete composition and the chloride adsorption isotherms for the type of cement to be used. These parameters influence the physical and chemical binding capacity of the concrete along with the pore volume, which has to be saturated to the point where the chloride concentration in the pore solution is in balance with the exposure environment.

With C_{eqv} and the material parameters known, the chloride saturation concentration $C_{\text{S},0}$ can be calculated. Figure 28 shows the correlation between $C_{\text{S},0}$ and C_{eqv} for Portland cement concrete (c = 300 kg/m^3, w/c = 0.50). As can bee seen, for $C_{\text{eqv}} = 30$ g/l, $C_{\text{S},0}$ was determined to be 2.78 wt.-%/cement.

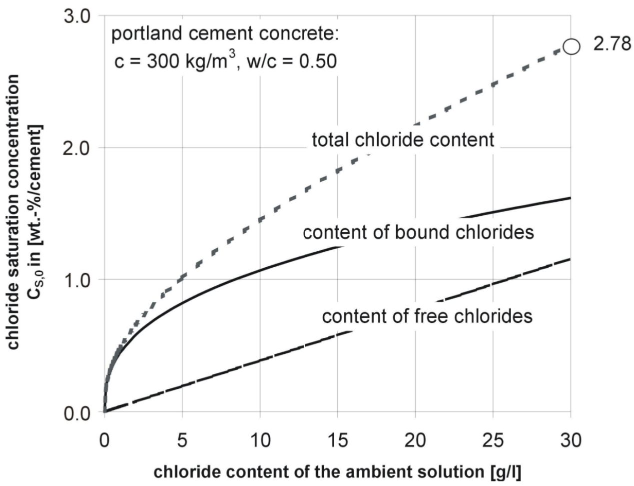

Figure 28: Correlation between $C_{\text{S},0}$ and C_{eqv} for Portland cement concrete (c = 300 kg/m^3, w/c = 0.50 (*fib* Bulletin 34 2006).

In cases where a structure is loaded with constant concentration of chlorides, such as when concrete is continuously exposed to sea water, $C_{\text{S},0}$ on the surface of the concrete is often reached in a relative short time periods compared to the design service life ($C_{\text{s}} = C_{\text{S},0}$). Therefore, in those cases, the chloride content C_{s} at the surface of the concrete can be assumed to be constant with time from the beginning.

The transfer function Δx is used in cases where the structural elements are exposed to a solution of constant or varying chloride concentrations at irregular intervals. A structural element which is intermittently loaded with a chloride concentrations, interrupted by dry periods of air storage, where the water in the concrete close to the surface evaporates, any subsequent re-wetting provokes the process of capillary suction. This capillary suction leads to a rapid transport of chlorides into the concrete. The chlorides penetrate up to a depth Δx where the chlorides can accumulate with time until they create a saturation concentration $C_{\text{S},\Delta\text{x}} = C_{\text{S},0}$. The transfer function can be described by beta distribution. Under splash conditions, the distance where chlorides can rapidly penetrate can be limited to 6.0 mm $\leq \Delta x \leq$ 11.0 mm. In the spray zone, where the distance to the road surface is larger than 1.5 m, $\Delta x = 0$ because the formation of a convection zone cannot be detected anymore. For structures that are continuously submerged, C_{s} is considered to be equal to $C_{\text{S},0}$. In this case, no transfer function is needed, thus $\Delta x = 0$. In tidal conditions, Δx has to be quantified. The types of exposure conditions and the distribution types for Δx, are summarised in table 14.

Table 14: The different types of exposure conditions and the distribution type of Δx. Based on *fib* Bulletin 34 (2006).

	Distribution	**Condition**
Δx [mm]	beta	- splash (splash road environment, splash marine environment)
Δx [mm]	constant parameter, value: 0	- for submerged marine structures - for leakage due to seawater and constant ground water level - for spray conditions (spray road environment, spray marine environment
Δx [mm]	beta	- for leakage due to varying ground water level - for tidal conditions

The chloride contamination of an element in the splash- or spray zone increases with decreasing distance to the source of chlorides. $C_{S,\Delta x}$ is time-dependent, but in *fib* Bulletin 34 (2006) Model Code, its considered independent of time for simplification. In the Model Code, an equation is provided for the determination of the maximum chloride content in the concrete, C_{max}, that was derived for specific conditions (location in Germany, concrete type, time of exposure). For structures of other exposure or concrete mixes, an equivalent equation should be determined.

Under splash conditions, $C_{\mathrm{S},\Delta \mathrm{x}}$ is considered equal to C_{max}. In the spray zone, C_{max} equals C_{s}. For these two exposure conditions, $C_{\mathrm{S},\Delta \mathrm{x}}$ resp. C_{s} can be assumed to follow normal distribution:

$$C_{\mathrm{S},\Delta \mathrm{x}} \text{ resp. } C_{\mathrm{s}} \sim \mathcal{N}(\mu_{\mathrm{C}_{\mathrm{S},\Delta \mathrm{x}} \text{ resp. } C_{\mathrm{s}}}, \sigma_{\mathrm{C}_{\mathrm{S},\Delta \mathrm{x}} \text{ resp. } C_{\mathrm{s}}}) \tag{51}$$

In case of submerged structures, C_{s} can be considered to be equal to $C_{\mathrm{S},0}$.

Critical Chloride Content C_{crit}

In *fib* Bulletin 34 (2006), the critical chloride content is defined as 'The total chloride content which leads to the depassivation of the reinforcement surface and initiation of iron dissolution, irrespective of whether it leads to visible corrosion damage on the concrete surface'. A restricted distribution such as the beta distribution, with a lower bound of $C_{\mathrm{crit,min}} = 0.2$ wt.-%/cement (by weight of cement) can be used to describe C_{crit}.

6.3 Summary

Service life models can be used as a tool when designing for durability. Several models have been developed, among them the *fib* Bulletin 34 (2006). The *fib* Bulletin 34 (2006) contain several approaches to service life design. The design approaches are categorised as full probabilistic, partial factor, deemed to satisfy, and avoidance of deterioration. In principle, the models avoid or minimise deterioration caused by environmental actions and can be applied for the design of new structures and for the assessment of existing structures.

Various deterioration mechanisms are treated in *fib* Bulletin 34 (2006), where they are quantified with models and their acceptance criteria is selected. One of those is the fully probabilistic design approach for the modelling of carbonation induced corrosion, which is presented in this chapter. It is based on the depassivation limit state where the concrete cover and carbonation depth at time t are compared. The carbonation depth is modelled taking into account various of parameters related to the environment and material. The parameters are quantified in the model code. Similar to carbonation induced corrosion, a full probabilistic design approach for the modelling of chloride induced corrosion of uncracked concrete is presented. The model is based on a depassivation limit state where the critical chloride content is compared to the actual chloride concentration at the depth of the reinforcement steel at time t. As with the model for carbonation induced corrosion, the critical chloride content is modelled taking into account various of parameters related to the environment and material that are quantified in the model code.

7 Assessment of an Existing Bridge

7.1 Structural Safety of Osvold Bridge

7.1.1 Introduction

Osvold bridge is a simply supported reinforced concrete slab bridge located on national road 85 in Sortland municipality, Nordland county, in northern Norway. The bridge was built in 1963 according to load regulation 1/1958 provided by NPRA (NPRA 2003). In Brutus, the current classification is as follows:

- Bk 10/60 - Road group A;
- Sv 12/65;
- Sv 12/100 - Centric.

The bridge consists of a concrete slab with underlying edges simply supported on concrete abutments. For the classification, the reinforcement in the slab is based on bridge standard 1/1958 (NPRA 2003) - reinforced concrete slab bridge, with driving space $7.5 - 8.0$ m, 10 m free length, load classification 1/58 (axle load 13t) (NPRA 2021*b*).

The last simple inspection on the bridge was conducted in 2021 and the last main inspection in 2020. Reinforcement corrosion was detected on the bottom side of the bridge slab, first in 2000, and has since then been registered in Brutus after regular inspections. The damage was last registered in Brutus after the simple inspection in 2021. The damages are cracks in the concrete cover and spalling in several local, approximately 100x100 mm, areas. Visible corroded reinforcement was also registered. The cause of damage was registered as carbonation. The damage degree was classified as level 2 damage and the damage consequence as 2B, meaning it has medium consequence on the load-carrying capacity, and 2V, meaning it has medium consequence on maintenance cost, according to the damage assessment procedure at NPRA, described in section 4. In Brutus, the next simple inspection is planned to be carried out in 2022 and main inspection in 2025 (NPRA 2021*a*).

7.1.2 Capacity Assessment by Norwegian Public Roads Administration

In 2021, a capacity assessment (NPRA 2021*b*) of Osvold bridge was conducted according to manuals V412 (NPRA 2021*c*) and V413 (NPRA 2021*d*), which are in appendix E. The method is based on the Eurocode semi-probabilistic method where partial safety factor format is used (described in appendix B.4.3). The classification included checking of the moment capacity in the longitudinal direction of the concrete slab, including the edges. The bridge was checked for the current traffic loads classification in Brutus.

As previously mentioned, scaling and cracking had taken place on the underside of the slab. However, the calculations made in the capacity assessment did not take into account the reduction of the reinforcement cross section due to corrosion. It is mentioned that final classification should take into account reduction of the reinforcement cross section due to corrosion. In addition, the classification does not include the assessment of foundations at Osvold bridge.

The Basis for Capacity Calculations

The calculation program *Sofistik* was used to find the design moments for the various traffic loads. The program *G-prog Concrete Analysis* was used to find the capacity of the concrete cross sections for the design moment. Manuals V412 and V413 along with Eurocode standards were used as a basis in the classification.

Model

The bridge model in *Sofistik* is made up of slab elements. The geometry used in the model was based on load regulation 1/1958 and the bridge drawing.

Materials

The year of construction registered in Brutus and handbook V413 were used as a basis for determining material qualities used in the model and calculations. The following parameters were used:

- Concrete;
- B20 strength class, concrete;
- Material factor, concrete - 1,5;
- Ks40 (380 MPa) - yield strength reinforcement, 25 – 32 mm;
- Material factor, reinforcement - 1,25.

Loads and Load Combination

The following loads were used for the calculations in *Sofistik*:

- Self-weight concrete 25 kN/m^3 - modelled geometry;
- Self-weight wear layer - 20 kN/m^2 (800 mm asphalt);
- Concrete edge - 4,0 kN/m - inserted as line load 15 cm from slab edge;
- Railings - 0,5 kN/m - inserted as line load 15 cm from slab edge;
- Traffic loads (imposed loads, Sv 12/65, Sv 12/100).

The traffic loads are placed as eccentric as possible and as much as possible in the middle of the span. The traffic loads for special transport Sv 12/100 are placed centrally with eccentricity in accordance with manual V412 and as much as possible in the middle of the span. For the axle loads, it is assumed that the load surface is spread in a ratio of 1:1 to the center of the concrete slab and in a ratio of 2:1 through the wear layer. The design loads are calculated in *Sofistik* where the load factors from figure 47 were used.

Design Check

With the loads and assumptions mentioned above, the utilisations (NO: Utnyttelse) for the slab and edges were calculated (NPRA 2021*b*). The calculation results can be seen in table 15.

Table 15: Utilisation for traffic loads and 800 mm wear layer (NPRA 2021*b*).

Traffic load		**Utilisation**
Imposed load - Bk 10/60	Slab (longitudinal direction)	1.31
	Edges	1.45
Motorised equipment - SV 12/65	Slab (longitudinal direction)	1.31
	Edges	1.50
Special transports - SV 12/100	Slab (longitudinal direction)	1.22
	Edges	1.30

As can be seen, the capacity of the bridge cannot be verified for any of the traffic loads, as the design moment is larger than the capacity of the bridge (utilisation > 1). In the assessment, the allowed wear layer thickness was calculated for each of the traffic loads. The results along with utilisations are shown in table 16.

Table 16: Traffic loads utilisation for allowed wear layer (NPRA 2021*b*).

Traffic load		**Utilisation**	**Allowed wear layer thickness [mm]**
Use load - Bk 10/60	Slab (longl. dir.)	0.85	100
	Edges	0.97	100
Motorised equipment - SV 12/65	Slab (longl. dir.)	0.83	50
	Edges	0.99	50
Special transports - SV 12/100	Slab (longl. dir.)	0.93	350
	Edges	0.99	350

Reflections on the Capacity Assessment

The semi-probabilistic method used for load-carrying capacity assessments at NPRA is a practical approach but can sometimes be excessively conservative. The partial safety factors are intended to cover a large number of uncertainties, which leads to the conclusion that they might not be very representative of the actual condition of the structure, especially in the case of ageing or damaged structures (such as in the case of Osvold bridge). More sophisticated probabilistic methods, where all influencing variables and their variability is taken into account, and where deterioration is considered, might therefore be necessary to verify the load-carrying capacity of structures.

7.2 Illustrative Example - Probabilistic Inspection Update

7.2.1 Introduction

Prediction models, such as the ones described in section 6, can be used in order to verify the safety of a structure and to facilitate maintenance planning. Taking deterioration into account in the determination of load-carrying capacity of a structure can be done in various ways, for example by adapting limit states to take into account damage or deterioration.

For existing structures, relevant information is found from historical data and documents, inspection data, weather stations, and so on. Based on these parameters, prediction can be carried out. The results give information on the failure probability of the considered structure or structure element over its service life. Decisions regarding maintenance measures can made at this point. To improve the accuracy of the prediction model, the calculation can be updated with data which is obtained from an inspection of the structure. Based on the updated model, decisions on further maintenance measures are made.

7.2.2 Decision Problem

In this example, the application of the approach described above is presented for an existing bridge subject to carbonation. The propagation of the carbonation front is predicted using a depassivation model presented in *fib* Bulletin 34 (2006). The carbonation front is compared with the concrete cover depth of the structure. The time dependent probability of failure is calculated with a Monte Carlo model. The Monte Carlo method is further described in appendix B.4.4. The code used for the calculations can be found in appendix F. The model is updated with inspection data after 20 years of exposure. In this example, the updated information involves the analysis of the concrete composition, which affects the quantification of the inverse carbonation resistance used in the model. Based on the updated model, decisions regarding maintenance measures are made.

7.2.3 Eurocode Reliability Requirement

To find out if and when the bridge needs maintenance measures, it is assumed that if the requirements in regard to reliability of the structure are not met, then maintenance measures are required. The requirements are provided by EN 1990 (2002), see fig. 29. The minimum target reliability index for a servicability limit state for a reference period of 50 years is $\beta = 1.5$, which corresponds to failure probability $p_f = 7\%$. This applies for resistance class RC2, which is assumed to be the case for the structure in this illustrative example.

Table C2 - Target reliability index *β* for Class RC2 structural members [1)]

Limit state	Target reliability index	
	1 year	50 years
Ultimate	4,7	3,8
Fatigue		1,5 to 3,8 [2)]
Serviceability (irreversible)	2,9	1,5

[1)] See Annex B
[2)] Depends on degree of inspectability, reparability and damage tolerance.

Figure 29: Target reliability index β for Class RC2 structural members (EN 1990 2002).

7.2.4 Carbonation Depth Prediction and Full Probabilistic Calculation

The propagation of the carbonation front is estimated with eq. (35). All parameters used in the formula, with the exception of the cover depth a, relative humidity RH_{real}, time of wetness ToW, probability of driving rain p_{SR}, and inverse carbonation resistance $R^{-1}_{\mathrm{ACC,0}}$, are extracted from *fib* Bulletin 34 (2006). The cover depth a would be found from tender documents while RH_{real}, ToW, and p_{SR} would be retrieved by evaluating data from a weather station (in this example they are hypothetical). For the determination of the inverse carbonation resistance $R^{-1}_{\mathrm{ACC,0}}$, the ACC-test method, described in section 6.2.1 would be used, however, it is assumed that no test data is available in this hypothetical example. In this case, literature data provided by *fib* Bulletin 34 (2006), shown in fig. 30 and fig. 31, can be used as an alternative to the test.

$R_{ACC,0}^{-1}$ [10^{-11} (m²/s)/(kg/m³)] / cement type	w/c$_{eqv.}$ [1]					
	0.35	0.40	0.45	0.50	0.55	0.60
CEM I 42.5 R	n.d.[2]	3.1	5.2	6.8	9.8	13.4
CEM I 42.5 R + FA (k = 0.5)	n.d.[2]	0.3	1.9	2.4	6.5	8.3
CEM I 42.5 R + SF (k = 2.0)	3.5	5.5	n.d.[2]	n.d.[2]	16.5	n.d.[2]
CEM III/B 42.5	n.d.[2]	8.3	16.9	26.6	44.3	80.0

[1] equivalent water cement ratio, considering FA (fly ash) or SF (silica fume) with the respective k-value (efficiency factor).

The considered contents were: FA: 22 wt.-%/cement; SF: 5 wt.-%/cement.

[2] n.d. – inverse effective carbonation resistance $R_{ACC,0}^{-1}$ has not been determined for these concrete mixes.

Figure 30: Quantification of $R^{-1}_{\mathrm{ACC,0}}$. Taken from *fib* Bulletin 34 (2006).

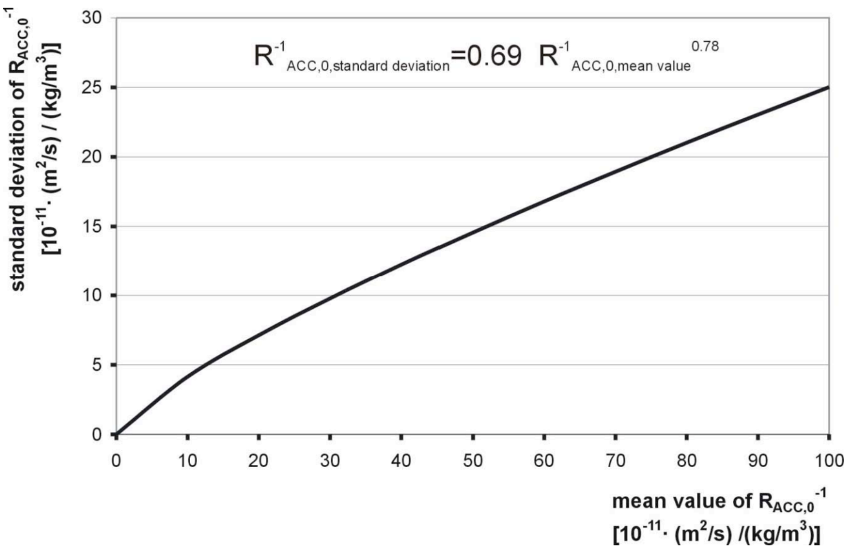

Figure 31: Quantification of $R^{-1}_{\mathrm{ACC},0}$; determination of the standard deviation based on the mean value. Taken from *fib* Bulletin 34 (2006).

In the example, the cement type of the concrete is assumed to be of type CEM I 42.5 R. With this information, fig. 30, and fig. 31, the inverse carbonation resistance is found. The variables used for the modelling are quantified with mean values μ and corresponding standard deviation σ. All parameters are listed in table 17.

Table 17: List of parameters needed for the depassivation model.

Parameter	Source	Unit	Distribution	μ	σ
a	tender documents	mm	normal	35	10
RH_{ist}	weather station	%	normal	70.0	11.2
RH_{ref}	fib model code	%	constant	65.0	-
g_{e}	fib model code	-	constant	2.5	-
f_{e}	fib model code	-	constant	5.0	-
b_{c}	fib model code	-	normal	-0.567	0.024
t_{c}	fib model code	days	constant	3.0	-
k_{t}	fib model code	-	normal	1.25	0.35
$R^{-1}_{\text{ACC,0}}$	fib model code	$(\text{mm}^2/\text{years})/(\text{kg}/\text{m}^3)$	normal	4225.8	1646.2
ϵ_{t}	fib model code	$(\text{mm}^2/\text{years})/(\text{kg}/\text{m}^3)$	normal	315.5	48
C_{s}	fib model code	kg/m^3	normal	0.00082	0.0001
ToW	weather station	-	constant	0.20	-
b_{w}	fib model code	-	normal distribution	0.446	0.163
p_{SR}	weather station	-	constant	0.0	-
t_0	fib model code	years	constant	0.0767	-

Based on the mean values of the quantified parameters it is possible to predict the development of carbonation depth with equation eq. (35). The results are given in fig. 32.

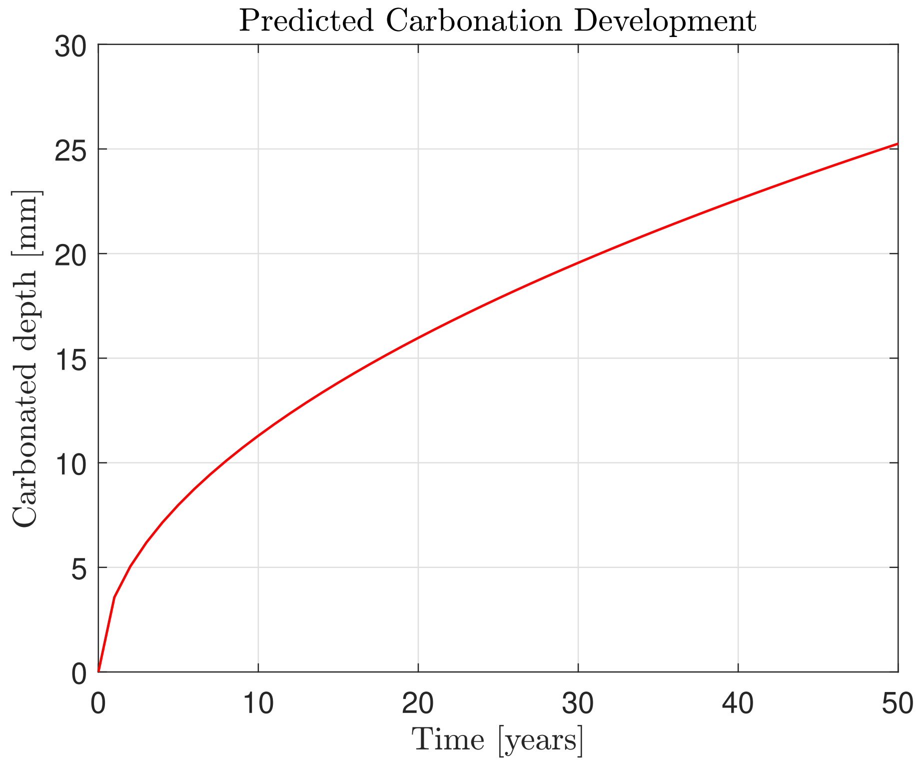

Figure 32: Predicted development of carbonation, calculated according to eq. (36) with the mean values and standard deviations from table 17.

The limit state considered in the example describes the probability that depassivation is initiated, by comparing the concrete cover with the carbonation depth. The limit state is given by eq. (36). The full probabilistic service life prediction calculations was calculated with the Monte Carlo method, taking into account all load and resistance variables with their variability into account. The number of trials in the calculations is $N = 10^6$. The time dependent increase of the predicted failure probability p_f, and the decrease of reliability index β over 50 years of exposure time, is illustrated in fig. 33. It is worth to note that the probability of failure p_f means, in this case, that depassivation is initiated, but not failure of the structure.

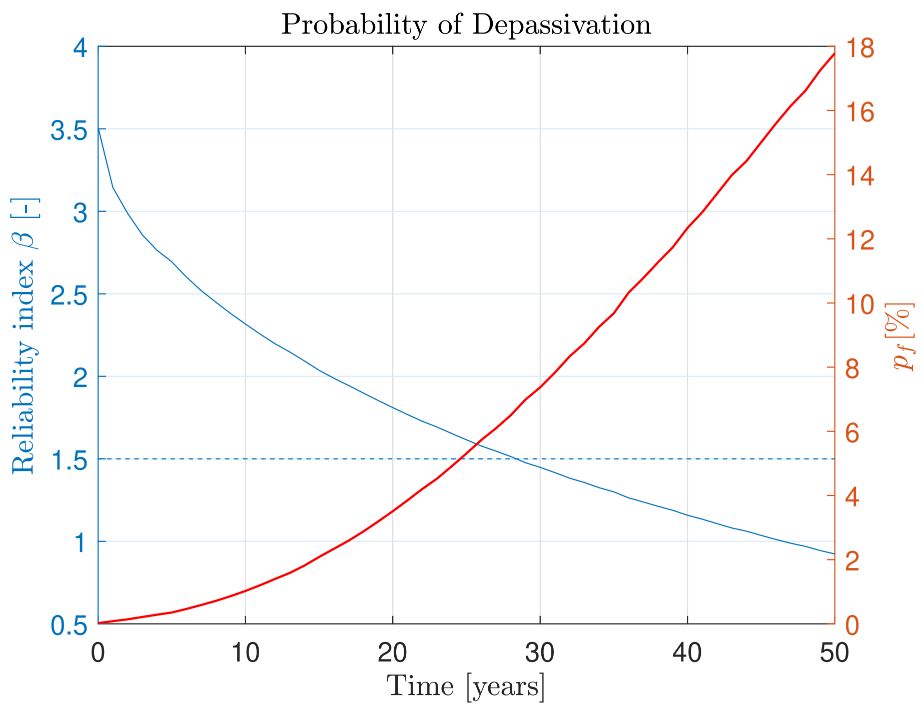

Figure 33: Time dependent probability of failure p_{f} and corresponding reliability index β.

7.2.5 Updating Deterioration Model with Inspection Data

For the purpose of illustration, it is now assumed that at year 20, inspection has been conducted and a test performed to figure out the composition of the concrete. From the test it was found that the cement type is CEM I 42.5 R + FA (k=0.5). With this information, the mean value and standard deviation for the inverse carbonation resistance can be updated by taking the corresponding values for this cement type from fig. 30, and fig. 31:

$$\mu_{\mathrm{R_{ACC,0}}^{-1}} = 3090.5\ (\mathrm{mm}^2/\mathrm{years})/(\mathrm{kg}/\mathrm{m}^3);$$

$$\sigma_{\mathrm{R_{ACC,0}}^{-1}} = 1299.3\ (\mathrm{mm}^2/\mathrm{years})/(\mathrm{kg}/\mathrm{m}^3).$$

Taking the inspection data into consideration, the deterioration model can be updated. The calculated failure probability of the updated model is shown in fig. 34.

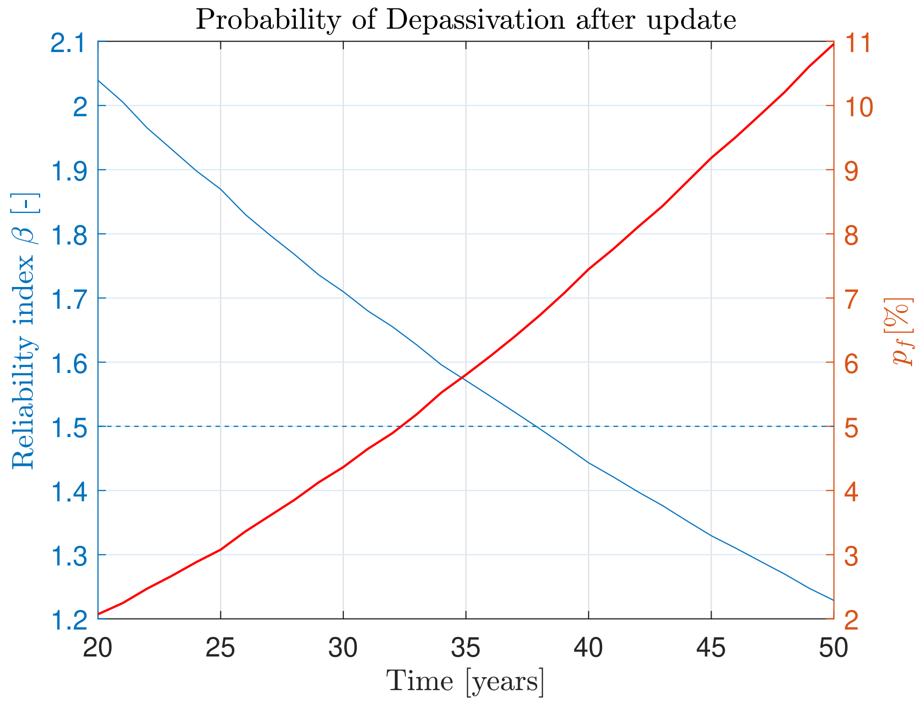

Figure 34: Updated time dependent probability of failure p_f and corresponding reliability index β.

7.2.6 Assessment

The results from the prediction model before and after the inspection update can be compared, see fig. 35.

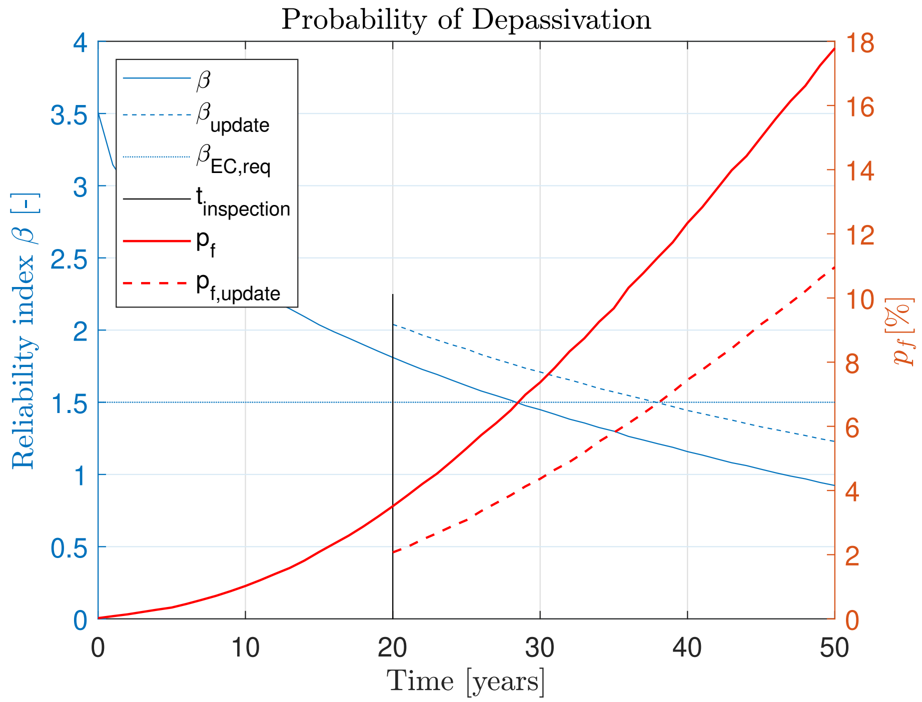

Figure 35: Updated time dependent probability of failure p_f and corresponding reliability index β.

From fig. 35 it can be observed that before the update, the reliability index reaches the minimum target reliability index ($\beta = 1.5$) at year 28. This implies that a maintenance measure is necessary before year 28. After the update, the the reliability index reaches the minimum target reliability index at year 38. The reason for this difference is that the parameters used in the first model may be conservative assumptions. By performing measurements, more accurate data from the structure is obtained, in this example the uncertainty of the inverse carbonation resistance was reduced, which in turn provides a more accurate prediction model.

Comparing the results it can be concluded that maintenance measures can be delayed by 10 years, which is favourable for the bridge owner as it would save resources allocated for maintenance.

7.3 Summary

A load-carrying capacity assessment performed by NPRA was presented in this section. The capacity assessment revealed that the load-carrying capacity of the bridge could not be verified. In the assessment it was declared that the corrosion damage is not included in the capacity assessment. By reducing the thickness of the wear layer, the bridge capacity can be verified.

An example of a full probabilistic approach for the service life modelling of carbonation induced corrosion for an existing bridge was presented. In addition, the model was updated with the incorporation of inspection data. The predicted probability of failure decreased with the update and the corresponding reliability index increased.

The load-carrying capacity assessment and the service life modelling example are useful for different purposes and are therefore not comparable. A 'complete' probabilistic approach would be a structural reliability analysis of the load-carrying capacity, adapted in such a way that the reduction of reinforcement would be considered, for example from service life modelling. Additionally, it is important to note that when damages can directly be observed, this type of service life model would not be beneficial. In those cases, the damages are registered during an inspection and the structure can be repaired accordingly. However, in cases where the damage is not visible, this type of modelling would enable early detection of damages. The models provide basis for long term maintenance planning and decision makers can predict the need for maintenance measures, leading to possibly cheaper mitigation measures. For Osvold bridge in particular, this type of modelling would not be very beneficial at this point in its service life, as reinforcement corrosion is already visible. Had this type of modelling been done earlier in the lifetime of the bridge, before any damages where visible, the progression of the corrosion could possibly have been prevented by implementing protective measures or small repairs and/or maintenance.

8 Discussion

This chapter includes discussion on the findings obtained from the previous chapters.

8.1 Decision Making

To support decisions, probabilistic methods can be used. Decision analysis attempts to guide the decision maker in situations, typically in the face of uncertainty, where one or more decisions are to be made. An example was given in section 3, to illustrate the process of selecting the optimal inspection and repair strategy. The results show that the Bayesian decision approach is an effective method to find an optimal solution in a consistent way.

The decision maker can gather all possible information available in order to make a high quality decision but this would also result in high cost and effort. With the Bayesian decision making approach, the decision maker can determine which information is necessary to make the optimal decision. The decision analysis approach can be used to inform the decision maker about the value of gathering more information before choosing an alternative.

8.2 Bridge Inspection Practises

Bridge inspections are a crucial part of maintaining and ensuring the safety of bridges. Analysing the present condition and future development of deterioration leads to the determination of repair and maintenance strategies for the bridge.

The manual V441, which NPRA has developed and is intended to be followed during inspections of existing bridges in Norway, gives a good overview and simple guidance of the inspection processes of bridges. The handbooks are easy to follow and the approach is simple and practical. Additional to the handbooks is the bridge management system Brutus, consisting of a database that holds all relevant information on each bridge. The system is user-friendly which is crucial as it provides the foundation for determining the repair and maintenance activities on the bridges. Even though the inspection method is rational, there are some problems that can be identified. One of the most important part of an inspection process is the inspector himself and the evaluation he does after an inspection. With the method NPRA uses, there is no clear threshold for the different classifications of damage degree and the degree of consequence. As a result, two inspectors, who follow the same handbooks and regulations for an inspection of a bridge, will most likely end up with somewhat different reports. In addition, the decision whether to perform additional measurements or material testing, and how many measurements are sufficient to make a decision, may not always be clear. This can lead to inconsistency in decision making regarding further management and maintenance of the bridge. By using Bayesian decision analysis approach in inspection practises, the decision making becomes more consistent and can be optimised.

According to manual V441 NPRA (2019), bridges in Norway are inspected at fixed calendar intervals without regard to the condition of the bridge. This approach results in the same inspection intervals for bridges in varying conditions without considering factors such as the age of the bridge, its condition, environment, and so on. Some bridges are old, in poor condition and thus in need of inspection and condition assessment, while others may be younger, in good condition, and unlikely to deteriorate, thus not in need of an inspection in the coming years. As NPRA has a large inventory of bridges, this can lead to inspection resources not being effectively utilised. Even if the inspections are always carried out as planned and according to regulations, like Hagstrøm (2022) stated in the interview in section 5, with such a large stock of bridges throughout the country, it must be difficult to keep up to date the inspections if there are resource and budget constraints.

The addition of ROS-analyses has a positive impact since it can reduce the waste of time and save resources, as in many cases the inspections can be considered unnecessary. ROS-analysis can be used in the assessment of bridges to identify the bridges for which the inspection intervals can be shortened or prolonged. It is also a good tool to gain understanding about the bridge condition and to map out possible undesirable events and vulnerabilities. Currently, ROS-analysis is not implemented for all bridges maintained by NPRA and no official guidelines exist for the analyses. ROS-analysis should be a routine linked to the inspection processes. That way, every bridge would at least be considered to be evaluated and have the possibility of delaying inspections when it is not necessary. The ROS-analysis should be verified, i.e. that they assess relevant risks. Solid understanding of the structural risks in Norwegian conditions would improve the method of doing ROS-analysis.

8.3 Service Life Prediction

The prediction of the time that a bridge or a bridge element is in need of repair or maintenance can guide decision making in bridge management. While inspecting deterioration is a part of condition assessments of bridges, taking deterioration into account in structural analysis is not a standard procedure at NPRA.

Predictive deterioration model is a good tool for effective repair and maintenance decision making. The benefits of being able to predict the future development or condition of a bridge or a bridge element include the most appropriate repair or maintenance strategy being performed at the right time during the service life of the bridge. The predictive deterioration models can be used to plan preventive maintenance measures, where the main objective is to avoid expensive and extensive repair or maintenance work in the future. Non-visible damages would potentially be detected earlier and thus money would be spent on smaller maintenance instead of when the damage becomes critical and is in need of extensive repair. This would optimise the repair or maintenance budget for the bridge owner. From the interview presented in section 5, it was mentioned that NPRA has enough funds to do maintenance on critical damages (Hagstrøm 2022). If there is only enough money to fix critical damages, there must come a point in time in the future where there are critical damages on so many bridges that they don't have the funds or resources to fix them all, consequently jeopardising the safety of the bridges. In the interview it was also mentioned that it would be ideal that damages would be treated before they get critical. Using prediction models and focusing on preventive maintenance would be a good way to avoid critical damages and reduce maintenance and repair costs.

Taking account of deterioration in the determination of load-carrying capacity of bridges can be done in various ways, which also depend on the type of deterioration. The application of service life models also depend on the experience and subjective opinion of the engineer making the assessment, which is one of the reasons why it can be difficult to quantify the parameters used in the models. The chosen input data can have a large influence on the results of decision making. The data has to be accurate enough for decisions to be based on them. Lack of reliable data can lead to inaccuracy and can be an obstacle for utilising decision models. This is also true for inspection data used to update the models. Many of the parameters needed for service life models can be obtained through routine inspections. Using inspection results in assessments can give a more accurate prediction of the condition of the bridge. Reliable methods for obtaining inspection data are required. Often, it is not possible to use inspection results directly in assessments. Therefore, there may be the need to change inspection methods so that their results can be used directly in the assessments. Collecting inspection data systematically would be valuable for NPRA, as the data can be used to improve the quality of deterioration models. After some years, there would be a great inventory of inspection data that can be used for the models. Bayesian updating can be used for the improvement of deterioration models. The deterioration models are repeatedly updated as the inspection data is incorporated. The illustrative example in section 7.2 was presented to show how a service life model can be used to predict damage development. By updating the model by incorporating inspection data, it was discovered that planned maintenance measures could be delayed by a substantial amount of time after the update, meaning resources allocated for maintenance can be saved.

8.4 Assessing Existing Bridges

The method used by NPRA to evaluate the load-carrying capacity of bridges involves load and material strength manuals that are based on the Eurocode standards. There, a semi-probabilistic method is introduced through partial factor design. This method can be sufficiently accurate but such deterministic methods can lead to lack of accuracy and conservative results. When dealing with a large stock of bridges, it would be a waste of resource to follow same conservative method for all bridges. When classifying a bridge, the manuals V412 (NPRA 2021*c*) and V413 (NPRA 2021*d*) are used, for example to determine the dead load of concrete. This dead load is used in combination with a load factor, which covers for uncertainty of the parameter variability. Using measurements from inspections could reduce this load factor and thereby decrease the dead load of the structure significantly, affecting the load-carrying capacity classification.

As an alternative for the semi-probabilistic method, more detailed probabilistic methods may be used instead of using characteristic values in combination with partial safety factors. The probabilistic methods would take into account the load and resistance variables with their variability. This would be a more time consuming and expensive approach, however, with probabilistic methods, new information can be accounted for and limit states adapted to take deterioration into account. That would result in a more accurate and economic result.

An issue facing NPRA and similar organisations that manage bridges is that traffic loads are changing with increasing traffic demand. This is a problem for ageing structures since they were originally designed for lower traffic loads according to standards at that time. Thus, there is often the situation where bridges need to be reassessed for traffic loads. It can therefore be of importance that the method used for reassessment of bridges gives a more accurate result. For example, in a reclassification of a bridge, current assessment methods might result in a unsatisfactory classification. This was the case in the load-carrying capacity assessment of Osvold bridge, presented in section 7.1, where it was revealed that the capacity could not be verified. Consequently, it would lead to costs related to maintenance or replacement of the bridge. By implementing a probabilistic approach, the capacity of the same bridge might reveal that the load-carrying capacity is satisfactory. That would avoid possibly costly maintenance measures. In addition, the corrosion damage identified during an inspection could be included by adapting limit states to take damage or deterioration into account.

By incorporating inspection results into capacity assessments, more accurate prediction of the load-carrying capacity of the bridge can be obtained. Consequently, this would make a difference for the management of the bridge the same way as discussed previously when using service life models. As for service life prediction models, the results from a probabilistic assessment depends strongly on the assumptions made about the uncertainties associated with parameters. If these assumptions are not based on adequate data, they are not valid, and then the results of the assessment will be inadequate.

9 Conclusions and Recommendations for Further Work

This chapter brings together the final conclusions of the work described in the previous chapters along with recommendations for future work for decision making and bridge assessments.

9.1 Conclusions

The purpose of this thesis was to explore and illustrate the potential that a decision making framework concerning assessments of existing bridges in Norway would bring. In the study, an emphasis was put on reinforced concrete bridges subject to deterioration due to carbonation induced corrosion and chloride induced corrosion. With the bridge population ageing, problems associated with structural deterioration in existing structures are evident. The challenge of managing a large portfolio of bridges addresses the need for a systematic way for deciding on optimal methods and timing for the repair and maintenance of bridges. The key conclusions that can be drawn as a result of the study are presented in the following.

- Bayesian decision theory enables rational and coherent basis for decision making regarding bridge inspection, repair, and maintenance, when uncertainties are involved in the process. Using this method can lead to the optimisation of decision making.
- The inspection practices currently used by NPRA can be improved by implementing probabilistic methods. These methods show a good potential for guiding decision making in bridge inspections and assessments and reduces inconsistency. By having standardised risk and vulnerability analysis as a routine linked to the inspection of every bridge in the system, inspection intervals can be extended or delayed, making resource management more effective.
- Predictive deterioration models can be used to forecast the need for maintenance. Their benefits include that the most appropriate repair or maintenance strategy is performed at the right time during the service life of the bridge. Parameters used in the service life models can be obtained through routine inspections and repeatedly incorporated in the models with Bayesian updating.
- The method used by NPRA for load-carrying capacity assessments can lead to lack of accuracy and conservative results, consequently leading to possibly unnecessary cost used for maintenance. Instead, more detailed probabilistic methods may be used, where inspection results can be accounted for and limit states adapted to take deterioration into account.

9.2 Recommendations for Further Work

The results from this thesis set out the potentials that a decision making framework would bring for the assessments of existing bridges. However, further research is required before such a framework can be set. The recommendations provided from this work are as follows:

- Adopt Bayesian decision analysis approach to current practises for the optimisation of inspection, repair, and maintenance of existing bridges. The simple concepts of the approach should be studied;
- Have verified risk and vulnerability analysis as a routine procedure linked to the inspections of bridges at the Norwegian Public Roads Administration;
- Use predictive service life models to guide decision making in bridge management. The models would be used as a basis for the determination of repair and maintenance strategies. For the models to give realistic results, the selection of input parameters needs to be evaluated. In addition, further work needs to be done on modelling the service life of deteriorating structures for multiple deterioration mechanisms;
- Explore the option of using a combination of semi-probabilistic methods and reliability based methods based on a probabilistic approach, in the assessment of existing structures. The probabilistic methods can be used as an additional option. The models can be adapted to include deterioration. The use of more sophisticated models can be helpful and provide a more reliable basis for decision making;
- Incorporating inspection results into capacity assessment models and prediction service life models for the improvement of their accuracy. Reliable methods for obtaining inspection data are required. What kind of inspection data can be used for the updating of models needs to be investigated.

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A Probability Theory

A.1 Elements of Probability

A.1.1 Meaning of Probability

The concept of the probability of a certain event is subject to various meanings and definitions. One interpretation of probability is the quantification of an individual's judgement or beliefs about whether a specific outcome of an event is likely to occur. In this interpretation, the probability is a subjective concept and has no meaning outside of expressing one's degree of belief. Another and the most prevalent interpretation of probability among scientists is the 'frequency interpretation'. This interpretation views the probability of a given outcome of an experiment as the proportion of the experiments that result in the outcome. Regardless of which interpretation is given to probability, there is a consensus that they both lead to the same mathematical rules (Ross 2021).

A.1.2 Sample Space and Events

Although an outcome of an experiment cannot be determined with certainty in advance, the assumption can be made that all possible outcomes are known. The set of all possible outcomes of an experiment is known as the *sample space* of an experiment and is denoted by S.

Any subset E of the sample space S is called an *event*. In other words, an event is a set consisting of possible outcomes of the experiment. For any two events E_1 and E_2 of sample space S, a new event $E_1 \cup E_2$, can be defined. The event $E_1 \cup E_2$ is called the *union* of events E_1 and E_2 and consists of all outcomes that are either in E_1, in E_2, or in both.

Similarly, the event $E_1 \cap E_2$, consisting of all outcomes that are both in E_1 and E_2, can be defined. This event is called the *intersection* of E_1 and E_2. In cases where the event $E_1 \cap E_2$ does not contain any outcomes, the event is referred to as the null event, denoted by . If $E_1 \cap E_2 =$, then E_1 and E_2 are said to be *mutually exclusive*.

For any event E, the event E^C is the *complement* of E and consists of all outcomes in the sample space S that are not in E (Ross 2021).

A.1.3 Axioms of Probability

For an experiment having a sample space S, a probability $P(E)$ is assigned to each event E. This probability is in accord with the following three axioms (Ross 2021):

Axiom 1 the probability that the outcome of the experiment is contained in E is a non-negative number between 0 and 1.

$$0 \leq P(E) \leq 1 \tag{52}$$

Axiom 2 the probability of the whole sample space is equal to 1.

$$P(S) = 1 \tag{53}$$

Axiom 3 for any set of mutually exclusive events the probability that at least one of these events occurs is equal to the sum of their respective probabilities.

$$P\left(\bigcup_{i=1}^{\infty} E_{\mathrm{i}}\right) = \sum_{\mathrm{i}=1}^{\infty} P(E_{\mathrm{i}}) \tag{54}$$

The three axioms set an upper boundary for the probability of any event. As mentioned before, the events E and E^C are always mutually exclusive. Furthermore, $E \cup E^C = S$. Axiom 52 and 53 give

$$1 - P(S) = P(E \cup E^C) = P(E) + P(E^C) \tag{55}$$

Equivalently, we have:

$$P(E^C) = 1 - P(E) \tag{56}$$

A.1.4 Conditional Probability

Conditional probabilities are considered to be one of the most important concepts in probability theory. Conditional probabilities are used to calculate probabilities when partial information concerning the result of the experiment is known, or for the update of probabilities when additional information is obtained. The desired probabilities are then the conditional ones (Ross 2021).

The conditional probability of E_1 given that E_2 has occurred is defined as

$$P(E_1 \mid E_2) = \frac{P(E_1 \cap E_2)}{P(E_2)} \tag{57}$$

Equation (57) is only defined when $P(E_2) > 0$. By multiplying both sides of the equation with $P(E_2)$, it can be seen that:

$$P(E_1 \cap E_2) = P(E_2)P(E_1 \mid E_2) \tag{58}$$

The equality shown in eq. (58) is known as the *chain rule* of conditional probabilities (Koller & Friedman 2009), and is often useful in computing the probability of the intersection of events (Ross 2021). Generally, if $E_1, ...E_k$ are events, the chain rule can be written as follows (Koller & Friedman 2009):

$$P(E_1 \cap ... \cap E_k) = P(E_1)P(E_2 \mid E_1) \cdots P(E_k \mid E_1 \cap ... \cap E_{k\text{-}1}) \tag{59}$$

A.1.5 Bayes' Rule

From the definition of conditional probability and the fact that $P(E_1 \cap E_2) = P(E_2 \cap E_1)$, *Bayes' rule* may be derived (Koller & Friedman 2009):

$$P(E_2 \mid E_1) = \frac{P(E_2 \mid E_1)P(E_1)}{P(E_2)} \tag{60}$$

Equation (60) can be generalised as follows. Consider the mutually exclusive events $F_1, F_2, ..., F_n$, where exactly one of those events must occur, in other words:

$$\bigcup_{i=1}^{n} F_i = S \tag{61}$$

By writing that

$$E = \bigcup_{i=1}^{n} E \cap F_i \tag{62}$$

the events $E \cap F_i$ are mutually exclusive, resulting in:

$$P(E) = \sum_{i=1}^{n} P(E \cap F_i) = \sum_{i=1}^{n} P(E \mid F_i)P(F_i) \tag{63}$$

Equation (63) shows that $P(E)$ is equal to a weighted average of $P(E \mid F_i)$, each term being weighted by the probability of the event on which it is conditioned. Given that event E has occurred, the probability of event F_j also occurring can be found by eq. (64):

$$P(F_j \mid E) = \frac{P(E \cap F_j)}{P(E)} = \frac{P(E \mid F_j)P(F_j)}{\sum_{i\,=\,1}^{n} P(E \mid F_i)P(F_i)} \tag{64}$$

Equation eq. (64) is known as Bayes' formula, which can be interpreted as showing us how opinions about hypotheses held before the experiment should be modified by the evidence of the experiment (Ross 2021).

A.1.6 Independent Events

Generally, the conditional probability $P(E_1 \mid E_2)$ is not equal to $P(E_1)$. In cases where they are equal, E_2 has occurred and does not change the probability that E_1 occurs, and E_1 is said to be independent of E_2. An event E_1 is independent of event E_2 in P, denoted $P \models (E_1 \perp E_2)$, if $P(E_1 \mid E_2) = P(E_1)$ or if $P(E_2) = 0$.

It is not common to encounter two independent events. A more common situation is when two events are independent given an additional event. An event E_1 is conditionally independent of event E_2 given event E_3 in P, denoted $P \models (E_1 \perp E_2 \mid E_3)$, if $P(E_1 \mid E_2 \cap E_3) = P(E_1 \mid E_3)$ or $P(E_2 \cap E_3) = 0$ (Koller & Friedman 2009).

Furthermore, the probability that two conditionally independent events occur can be calculated with the chain rule, given by eq. (58):

$$P(E_1 \cap E_2 \mid E_3) = P(E_1 \mid E_3)P(E_2 \mid E_3) \tag{65}$$

A.1.7 Random Variables

A *random variable* is defined by a function that associates with each outcome in an event space Ω a value (Koller & Friedman 2009). Usually uppercase roman letters $X, Y, Z, ...$ are used to denote random variables. Lowercase letters $x, y, z, ...$ refer to values of random variables. Boldface type are used to denote sets of random variables. Thus, $\boldsymbol{X, Y, Z, ...}$ are typically used to refer to a set of random variables and $\boldsymbol{x, y, z, ...}$ denote assignments of values to the variables in these sets (Koller & Friedman 2009).

Random variables are said to be *discrete* if their set of possible values are either a finite sequence $x_1, ..., x_{\mathrm{n}}$, or an infinite sequence $x_1, ...$. Random variables that take on a continuum of possible values are said to be *continuous* random variables (Ross 2021).

A.1.8 Probability Distribution

For a discrete random variable X, the *probability mass function* $p_{\mathrm{X}}(x)$ describes the probability of X being equal to a specific real value x and is defined as

$$p_{\mathrm{X}}(x) = P(X = x) \tag{66}$$

A continuous random variable X has the *probability density function* $f_X(x)$, which satisfies

$$P(X \in B) = \int_{\mathrm{B}} f_{\mathrm{X}}(x)\, dx \tag{67}$$

for any set of real numbers (Ross 2021). The *cumulative distribution function* F_{X} of a random variable X is defined for any real number x and provides the probability $P(X \leq x)$. For a discrete random variable X, F_{X} can be expressed in terms of $p_{\mathrm{X}}(k)$ by

$$F_{\mathrm{X}}(x) = P(X \leq x) = \sum_{\text{all k} \leq x} p_{\mathrm{X}}(k) \tag{68}$$

The cumulative distribution function F_{X} for a continuous random variable is expressed by

$$F_{\mathrm{X}}(x) = P(X \leq x) = \int_{\text{-}\infty}^{\mathrm{x}} f_{\mathrm{X}}(t)\, dt \tag{69}$$

A.1.9 Joint Probability Distribution

The relationship between two or more random variables is often of interest. To specify the relationship between two random variables X and Y, the *joint cumulative probability distribution function* $F_{X,Y}(x, y)$ is defined as (Ross 2021):

$$F_{X,Y}(x, y) = P[X \leq x, Y \leq y] \tag{70}$$

If X and Y are both discrete random variables, the *joint probability mass function*, $p_{\mathrm{X,Y}}(x, y)$ is

$$p_{\mathrm{X,Y}}(x, y) = P(X = x, Y = y) \tag{71}$$

Since Y must take on some value y, the event X = x can be written as the union, over all possible values of y, of the mutually exclusive events $\{X = x, Y = y\}$. Using Axiom 3 (eq. (54)), it can be seen that

$$p_{\mathrm{X}}(x) = P(X = x) = \sum_{\mathrm{y}} P(X = x, Y = y) = \sum_{\mathrm{y}} p_{X,Y}(x, y) \tag{72}$$

Similarly,

$$p_{\mathrm{Y}}(y) = P(Y = y) = \sum_{\mathrm{x}} P(X = x, Y = y) = \sum_{\mathrm{x}} p_{\mathrm{X,Y}}(x, y) \tag{73}$$

For continuous random variables X and Y, the *joint probability density function* $f_{\mathrm{X,Y}}(x, y)$ is a non-negative function that satisfies

$$P((X, Y) \in C) = \iint_{\mathrm{(x,y)\,\in\,C}} f_{\mathrm{X,Y}}(x, y)\, dxdy \tag{74}$$

For every subset C in a two-dimensional plane. If X and Y are jointly continuous, they are individually continuous. Thus, the probability density function of X is

$$f_{\mathrm{X}}(x) = \int_{-\infty}^{+\infty} f_{\mathrm{X,Y}}(x, y) dy \tag{75}$$

and the probability density function of Y is

$$f_{\mathrm{Y}}(y) = \int_{-\infty}^{+\infty} f_{\mathrm{X,Y}}(x, y) dx \tag{76}$$

A.1.10 Conditional Probability Distribution

For discrete random variables X and Y, it is natural to define the *conditional probability mass function* of X given that $Y = y$, by

$$p_{\mathrm{X|Y}}(x \mid y) = P(X = x \mid Y = y) = \frac{P(X = x, Y = y)}{P(Y = y)} = \frac{p_{\mathrm{X,Y}}(x, y)}{p_Y(y)} \tag{77}$$

for all values of y such that $f_{\mathrm{Y}}(y) > 0$. If X and Y have a joint probability density function $f_{\mathrm{X,Y}}(x, y)$, the *conditional probability density function* of X, given that $Y = y$, is defined for values of y such that $f_{\mathrm{Y}}(y) > 0$, by

$$f_{\mathrm{X|Y}}(x \mid y) = \frac{f_{\mathrm{X,Y}}(x, y)}{f_{\mathrm{Y}}(y)} \tag{78}$$

Conditional densities allows for the definition of conditional probabilities of events associated with one random variable when the value of a second random variable is given. That is, if X and Y are jointly continuous, then, for any set A (Ross 2021):

$$P(X \in A \mid Y = y) = \int_{\mathrm{A}} f_{\mathrm{X|Y}}(x \mid y) dx \tag{79}$$

A.1.11 Expectation

One of the most important concepts in probability theory is the expectation of a random variable. For a discrete random variable X with the probability mass function p_X, then the *expectation* or *expected value* of X, denoted by $\mathbb{E}[X]$ is given as (Ross 2021):

$$\mu_X = \mathbb{E}[X] = \sum_{x} x p_X(x) \tag{80}$$

For a continuous random variable with probability density function f_X, the *expected value* is given as

$$\mu_X = \mathbb{E}(X) = \int_{-\infty}^{\infty} x f_X(x) dx \tag{81}$$

A.1.12 Variance

The variance of a random variable X gives an indication of its variation, or spread. For a random variable X, with mean value μ, the *variance* of X, denoted $\mathrm{Var}[X]$, is defined as

$$\mathrm{Var}[X] = \mathbb{E}[(X - \mu)^2] \tag{82}$$

where an alternative formula for the variance is given by

$$\mathrm{Var}[X] = \mathbb{E}[X^2] - \mu^2 \tag{83}$$

A.1.13 Standard Deviation

The standard deviation, σ_X, of a random variable X, is another way to measure the amount of variation or dispersion. It is defined as the square root of the variance

$$\sigma_X = \sqrt{\mathrm{Var}[X]} \tag{84}$$

A.1.14 Covariance

The *covariance* of two random variables X and Y, is an important indicator of the relationship between them. The *covariance* of two random variables X and Y, $\mathrm{Cov}(X, Y)$, is defined as

$$\mathrm{Cov}(X,Y) = \mathbb{E}[(X-\mu_{\mathrm{x}})(Y-\mu_{\mathrm{y}})] \tag{85}$$

where μ_{x} and μ_{y} are the mean values of X and Y, respectively. If X and Y are independent random variables, then $\mathrm{Cov}(X,Y) = 0$. The relationship between X and Y is indicated by the correlation between them, which can be described by a dimensionless quantity, $\mathrm{Corr}(X,Y)$:

$$\mathrm{Corr}(X,Y) = \frac{\mathrm{Cov}(X,Y)}{\sqrt{\mathrm{Var}(X)\mathrm{Var}(Y)}} \tag{86}$$

$\mathrm{Corr}(X,Y)$ always has a value between -1 and $+1$. In general, a positive value of $\mathrm{Cov}(X,Y)$ is an indication that Y tends to increase when X does. A negative value of $\mathrm{Cov}(X,Y)$ indicates that Y tends to decrease as X increases.

B Structural Reliability Analysis

B.1 Structural Performance

Structures should be designed, operated, maintained, and decommissioned in a way that they support societal functionality and enhance sustainable societal development during their service life. The structures should, with appropriate degree of risk and reliability, fulfil performance requirements. These performance requirements are listed in ISO2394 (2015) and are as follows:

- Function adequately under all expected actions throughout their service life, providing service and functionality;
- Withstand extreme and/or frequently repeated and permanent actions, as well as environmental exposures occurring during their construction, anticipated use, and decommissioning, providing safety and reliability with respect to damage and failures;
- Be robust such as not to suffer severe damage or cascading failure by extraordinary and possibly unforeseen events like natural hazards, accidents, or human errors; providing sufficient robustness.

Structural performance models can be established for these performance requirements. In general, different levels of detail for the assessment of the structural performance can be distinguished (Köhler & Mendoza 2020). In order to assess the structural performance of a structure, the possible responses should be assessed and divided into two domains consisting of desirable and undesirable states. The boundary between these domains is called the limit state (ISO2394 2015).

B.2 Limit State

The safety of a structure depends on the magnitude of the applied load and the strength and stiffness of the structure. Whether that response is considered satisfactory depends on the requirements that need to be satisfied. That could be safety of the structure against collapse, limitations on damage or deflections, and other criteria. These requirements can be defined as a limit state (Melchers & Beck 2018). Typical limit states for structures are given in table 18.

Table 18: Typical limit states for structures (Melchers & Beck 2018).

Limit state type	**Description**	**Examples**
Ultimate (safety)	Collapse of all or part of structure	Progressive collapse, instability, corrosion, fatigue, deterioration.
Damage (often included in above)	-	Excessive or premature cracking, deformation.
Serviceability	Disruption of normal use	Excessive deflections, vibrations, local damage.

Structural reliability deals with the calculation and prediction of the probability of limit state violation for a engineered structural system at any stage during its service life. Furthermore, it deals with the violation of the ultimate or safety limit states for the structure (Melchers & Beck 2018).

Limit states can generally be written as:

$$G(\mathbf{X}) = G(a_0, X_1, X_2, ...X_\mathrm{n}) > 0 \tag{87}$$

The X_i represents the random variables, describing both the problem and the requirements for a particular basis of assessment. The random variables can represent strength, loads, dimensions, or even abstract values (Schneider & Vrouwenvelder 2017).

Failure F can be defined by the failure condition as:

$$F = G(\mathbf{X}) \leq 0 \tag{88}$$

The probability of failure p_f can be determined by the following equation:

$$p_\mathrm{f} = P[G(\mathbf{X}) \leq 0] \tag{89}$$

B.3 Reliability

Reliability can be defined as the complement of the probability of failure p_f, where p_f is the chance that a particular, predefined event occurs:

$$r = 1 - p_\mathrm{f} \tag{90}$$

Reliability is quantifiable. Lack of reliability implies that a condition, with a certain probability, will not be fulfilled. That could for example be a structure not collapsing, existing deflections do not exceed specific values, or reinforcement bars not rusting prematurely (Schneider & Vrouwenvelder 2017).

B.4 Structural Reliability Methods

There exist many methods to solve equation 89. In the simplest case, the probability of occurrence of an event such as limit state violation is a numerical measure of the chance of its occurrence. The measure can either be obtained from measurements of the long-term frequency of occurrence of the event for generally similar structures, or it may be simply a subjective estimate of the numerical value. In probabilistic assessments, any uncertainty about a variable is taken into account explicitly (Melchers & Beck 2018). The following subdivision of methods is often used (Schneider & Vrouwenvelder 2017):

Level III: Limit state functions and distribution functions for the random variables are introduced without any approximations. Calculations are usually based on Monte Carlo simulation or numerical integration;

Level II: The amount of calculation efforts is reduced by adopting linerization techniques, usually the so-called First Order Reliability Method. The degree of accuracy may strongly depend on the details of the problem being assessed;

Level I: The variables X_{i} are introduced by one single value only, referred to as the design value. This method does not calculate the probability of failure but only checks whether some defined target level is attained or not. It is the basis for most design and assessment procedure in every day practise and is referred to as the semi-probabilistic level.

It is important to remember that p_{f} is a subjective probability. It is a matter of the degree of confidence in the statement that has been assessed could fail. The probability of failure depends very much on the information available to the person making the assessment (Schneider & Vrouwenvelder 2017). Written formally, p_{f} is a conditional probability, dependent on the state of knowledge of the person doing the assessment:

$$p_{\mathrm{f}} = P[G(\mathbf{X}) \leq 0|\mathrm{Info}] \tag{91}$$

It is assumed that the variables in limit state function are independent of each other. Correlations between variables can be difficult to determine and complicate the algorithm. This limitation is acceptable because if there is uncertainty, both extreme cases (complete correlation and no correlation) can be analysed seperately, compared, and the differences in the result assessed. However, computer programs do allow for correlations. Another limitation is that human error is not included in this kind of analysis. Probability of failure is conditional on the assumption that there are no errors in what is being analysed (Schneider & Vrouwenvelder 2017).

In the following, a a special case with a linear limit state and normal random variables, semi-probabilistic method, and Monte-Carlo method will be introduced.

B.4.1 Linear Limit State Functions and Normal Distributed Variables

A simple approach can be applied where all variables are normal distributed and the limit function is linear (Melchers & Beck 2018). Lets consider this to be the case. The resistance, R, and the loading, S, are normally distributed. The limit state function is defined as:

$$G(R, S) = R - S \tag{92}$$

The safety margin is also normally distributed and is defined as:

$$Z = R - S \tag{93}$$

The mean value of the safety margin Z can be calculated as:

$$\mu_Z = \mu_\mathrm{R} - \mu_\mathrm{S} \tag{94}$$

and the standard deviation of the safety margin can be calculated as:

$$\sigma_\mathrm{Z} = \sqrt{\sigma_\mathrm{R}^2 + \sigma_\mathrm{S}^2} \tag{95}$$

The reliability index is calculated as follows:

$$\beta = \frac{\mu_\mathrm{Z}}{\sigma_\mathrm{Z}} \tag{96}$$

The probability of failure can then be calculated as:

$$p_\mathrm{f} = P(R - S \leq 0) = P(Z \leq 0) = \Phi\Big(\frac{0 - \mu_\mathrm{Z}}{\sigma_\mathrm{Z}}\Big) = \Phi(-\beta) \tag{97}$$

where $\Phi()$ is the standard normal distribution function (zero mean and unit variance).

The random variable $Z = R - S$ is shown in figure 36, in which the shaded area is the failure region $Z \leq 0$ (Melchers & Beck 2018). If either or both of the standard deviations σ_{R} and σ_{S} are increased, the probability of failure, p_{f}, will increase. Similarly, if the difference between the mean values μ_{R} and μ_{S} is reduced, p_{f} increases.

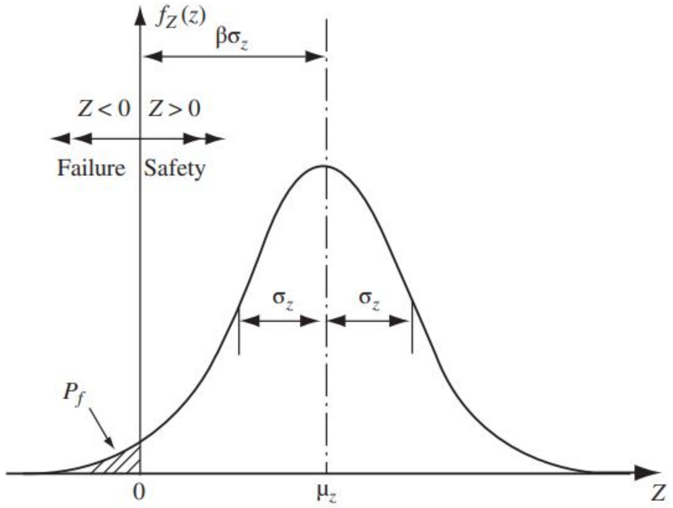


Figure 36: Distribution of Z (Melchers & Beck 2018).

B.4.2 Semi-Probabilistic Method

The semi-probabilistic method mentioned in appendix B.4 corresponds to the lowest level of detail. In this approach, a design decision is chosen such that it complies with the criterion that a design value of the resistance is larger than a design value of the load effect. The design value of the resistance R_{d} should have sufficiently low non-exceedance probability and the design value for the load S_{d} should have sufficiently low exceedance probability such that the design criterion in the limit ($R_{\mathrm{d}} = S_{\mathrm{d}}$) corresponds to the required level of reliability. In a load and resistance factor design (LRFD) format, design values are determined based on characteristic values and partial safety factors. Both, the definition of the characteristic value and the choice of partial safety factor, i.e., the reliability elements of the code, is made in order to meet the reliability requirements. This is generally referred to as code calibration (Köhler et al. 2019).

The principles of semi-probabilistic design are provided in ISO2394 (2015). A basic principle of the method is that semi-probabilistic safety formats should comprise:

- Consequence class categorisations;
- Design situations;
- Design equations;
- Design values.

The design equations should in general be formulated for failure modes involving in principle both failure of individual cross sections of the structures, as well as for failure modes involving the failures of several cross sections of the structures. The design values for the various action and material characteristics entering the design equations should account for the characteristics of the uncertainties associated with the loads and resistances, which are of relevance for the given design situation. The design value method takes basis in a direct check of the relevant design situations and corresponding design equations using design values for the basic variables which are determined on the basis of reliability assessment. The design values can be determined using simplified methods of direct use of First Order Reliability Method (FORM) (ISO2394 2015).

Semi-probabilistic design is the method of choice for most structural design decision problems and the executive guidance and standardisation can be found in national and international design standards, such as the Eurocodes (Köhler et al. 2019).

B.4.3 EUROCODE Semi-Probabilistic Design

In the Eurocode series of European Standards, EN 1990 (2002) establishes the principles and requirements for the safety, serviceability and durability of structures, describes the basis for their design and verification, and provides guidelines for related aspects of structural reliability. EN 1990 is intended to be used together with EN 1991-1999 for the design of structures. In addition, EN 1990 is applicable for the structural assessment of an existing construction, in developing the design of repairs and alterations, or in assessing changes of use.

EN 1990 provides a semi-probabilistic method used for design situations. A distinction should be made between ultimate limit states and serviceability limit states. The limit states should be related to design situations, which are selected taking into account the circumstances under which the structure is required to fulfil its function (such as persistent design situation and seismic design situation). The design for limit states shall be based on the use of the structural and load models for relevant limit states. Those requirements should be achieved by the partial factor method. The partial factor method is described in section 6 of EN 1990. An alternative design directly based on probabilistic methods may also be used.

In the partial factor method, the basic variables (i.e. actions, resistances, and geometrical properties) with the use of partial factors and other factors are given design values, and a verification is made to ensure that no relevant limit state has been exceeded. The general design equation can be expressed as:

$$E_{\mathrm{d}} \leq R_{\mathrm{d}} \tag{98}$$

where

E_{d} is the design value of the effect of actions;

R_d is the design resistance.

Design Values

The design values should be obtained by using the characteristic values, or other representative values, in combination with partial factors and other factors, which are defined in EN 1990 (2002) and in EN 1991-1999.

The design value of the effect of actions, E_d, can be expressed in general terms as:

$$E_\mathrm{d} = E_\mathrm{d}(\mathbf{F_d}; \mathbf{a_d}; \boldsymbol{\theta}_\mathbf{d}) \tag{99}$$

and the design resistance, R_d, can be expressed as:

$$R_\mathrm{d} = R_\mathrm{d}(\mathbf{X_d}; \mathbf{a_d}; \boldsymbol{\theta}_\mathbf{d}) \tag{100}$$

where

$\boldsymbol{F}_\mathbf{d}$ is a vector of design values of actions F;

$\boldsymbol{X}_\mathbf{d}$ is a vector of design values of material properties;

$\boldsymbol{a}_\mathbf{d}$ is a vector of design values of geometrical data;

$\boldsymbol{\theta}_\mathbf{d}$ is a vector of design values of model uncertainties.

Partial Factors

A design value $x_\mathrm{d,i}$ of any particular variable is given by either multiplying or dividing a characteristic value $x_\mathrm{k,i}$ with its corresponding partial safety factor γ_i (JCSS 2001). A characteristic value generally corresponds to a specified fractile of a assumed statistical distribution of the particular variable. In some circumstances a nominal value is used as the characteristic value (EN 1990 2002). For action variables, the characteristic values would be multiplied by the safety factor, and for the resistance variables, the characteristic values would be divided by the safety factor. Accordingly, the design value of the effect of actions (equation 99) can be written as $E_\mathrm{d} = E_\mathrm{k} \cdot \gamma_\mathrm{E}$, and the design resistance (equation 100) as $R_\mathrm{d} = \frac{R_\mathrm{k}}{\gamma_\mathrm{R}}$.

The partial factors account for uncertainty in representative values of actions, model uncertainty in actions and action effects, model uncertainty in structural resistance, and uncertainty in material properties (EN 1990 2002). The partial factors are an important element in controlling the safety of a structure designed to the code (JCSS 2001).

Reliability Requirements

A basic requirement in the Eurocode is that a structure shall sustain all actions and influences likely to occur during execution and use, with appropriate degree of reliability and in an economical way. In chapter 2.2(3) in EN 1990 (2002), the relevant factors that should be taken account of when choosing the levels of reliability for a structure are specified, including:

- The possible cause and/or mode of attaining a limit state;
- The possible consequences of failure in terms of risk to life, injury, potential economical losses;
- Public aversion to failure;
- The expense and procedures necessary to reduce the risk of failure.

The levels of reliability that apply to a particular structure may be specified by the classification of the structure as a whole, or by the classification of its components (see 2.2(4) of EN 1990 (2002)).

For the purpose of reliability differentiation, consequences classes (CC) may be established. This is described in Annex B.31 in EN 1990 (2002). The classification is built on the consequences of failure or malfunction of the structure. The consequence classes are shown in figure 37.

Table B1 - Definition of consequences classes

Consequences Class	Description	Examples of buildings and civil engineering works
CC3	**High** consequence for loss of human life, *or* economic, social or environmental consequences **very great**	Grandstands, public buildings where consequences of failure are high (e.g. a concert hall)
CC2	**Medium** consequence for loss of human life, economic, social or environmental consequences **considerable**	Residential and office buildings, public buildings where consequences of failure are medium (e.g. an office building)
CC1	**Low** consequence for loss of human life, *and* economic, social or environmental consequences **small or negligible**	Agricultural buildings where people do not normally enter (e.g. storage buildings), greenhouses

Figure 37: Definition of consequences classes as stated in EUROCODES (EN 1990 2002).

According to EN 1990 (2002) B3.1(2,3), the importance of a failure mode for the consequences should be considered. Different members of the structure can have different consequences class.

The reliability classes (RC) are defined by the β reliability index concept. The reliability classes may be associated with the consequences classes mentioned before. In B3.2 in EN 1990 (2002), minimum values for the reliability index are recommended. This is shown in figure 38.

Table B2 - Recommended minimum values for reliability index β (ultimate limit states)

Reliability Class	Minimum values for β	
	1 year reference period	50 years reference period
RC3	5,2	4,3
RC2	4,7	3,8
RC1	4,2	3,3

Figure 38: Recommended minimum values for reliability index β as stated in EUROCODES (EN 1990 2002).

Connection Between Reliability Analysis and Partial Safety Factors

In some cases, a direct correspondence between the design value and reliability requirements can be established by the design value method (Köhler & Mendoza 2020). To demonstrate this, one can look at the reliability problem from chapter B.4.1, where the reliability index was defined as:

$$\beta = \frac{\mu_\mathrm{Z}}{\sigma_\mathrm{Z}} = \frac{\mu_\mathrm{R} - \mu_\mathrm{S}}{\sqrt{\sigma_\mathrm{R}^2 + \sigma_\mathrm{S}^2}} \tag{101}$$

In the Eurocodes, the reliability requirement is given as a design requirement β_req. Therefore, equation 101 can be reformulated to facilitate the identification of a combination of R and S that comply with a specific β_req, such that $\beta \geq \beta_\mathrm{req}$. The distance between the mean values that is in compliance with the required reliability is determined as $\mu_\mathrm{R} - \mu_\mathrm{S} \geq \beta_\mathrm{req}\sigma_\mathrm{Z}$, whereas σ_z can be splitted into contributions from R and S as:

$$\mu_\mathrm{R} - \mu_\mathrm{S} \geq \beta_\mathrm{req}\frac{\sigma_\mathrm{R}}{\sqrt{\sigma_\mathrm{R}^2 + \sigma_\mathrm{S}^2}}\sigma_\mathrm{R} + \beta_\mathrm{req}\frac{\sigma_\mathrm{S}}{\sqrt{\sigma_\mathrm{R}^2 + \sigma_\mathrm{S}^2}}\sigma_\mathrm{S} \tag{102}$$

Introducing the weighting factors α_R and α_S as:

$$\alpha_\mathrm{R} = \frac{\sigma_\mathrm{R}}{\sqrt{\sigma_\mathrm{R}^2 + \sigma_\mathrm{S}^2}} \tag{103}$$

$$\alpha_\mathrm{S} = \frac{\sigma_\mathrm{S}}{\sqrt{\sigma_\mathrm{R}^2 + \sigma_\mathrm{S}^2}} \tag{104}$$

Separating the resistance from the load side, the following expression is obtained:

$$\begin{aligned} \mu_{\mathrm{R}} - \alpha_{\mathrm{R}}\beta_{\mathrm{req}}\sigma_{\mathrm{R}} &\geq \mu_{\mathrm{S}} + \alpha_{\mathrm{S}}\beta_{\mathrm{req}}\sigma_{\mathrm{S}} \\ \mu_{\mathrm{R}}(1 - \alpha_{\mathrm{R}}\beta_{\mathrm{req}}V_{\mathrm{R}}) &\geq \mu_{\mathrm{S}}(1 + \alpha_{\mathrm{S}}\beta_{\mathrm{req}}V_{\mathrm{S}}) \\ R_{\mathrm{d}} &\geq S_{\mathrm{d}} \end{aligned} \tag{105}$$

with the coefficients of variation V_{R} and V_{S} and the design values R_{d} and S_{d}. Equation 105 suggests the seperation of load and resistance values. However, this is not really the case as the weighting factors contain the standard deviations of both variables. To evaluate the weighting factors requires the full evaluation of the reliability problem (Köhler & Mendoza 2020). In order to circumvent this problem the EUROCODES introduce so-called standardized α values, which will not be further discussed in this section.

B.4.4 The Monte Carlo Method

An alternative procedure to calculate the probability of failure is the Monte Carlo method. The Monte Carlo method involves 'sampling' at 'random' to simulate artificially a large number of experiments and to observe the result. In the simplest approach of structural reliability analysis, that means sampling each random variable X_{i} randomly to give a sample value $\hat{x}_{\mathrm{i}}$. The limit state $G(\mathbf{x})$ is then checked using the sample set of values $\hat{x}_{\mathrm{i}}$. If $G(\mathbf{x}) \leq 0$, the structure or structural element has 'failed'. This is repeated many times, each time with a random vector $\hat{\boldsymbol{x}}$ of $\hat{x}_{\mathrm{i}}$ values (Melchers & Beck 2018). With N trials, the approximate probability of failure is:

$$p_{\mathrm{f}} \approx \frac{n(G(\hat{x}_{\mathrm{i}}) \leq 0)}{N} \tag{106}$$

where $n(G(\hat{x}_{\mathrm{i}}) \leq 0)$ is the number of trials n for which $G(\hat{x}_{\mathrm{i}}) \leq 0$. The greater the number of N trials, the more reliable is the value of p_{f}. This becomes clear when looking at the coefficient of variation for p_{f} (Schneider & Vrouwenvelder 2017). The coefficient of variation for small p_{f} can be written as:

$$v_{\mathrm{p_f}} \approx \frac{1}{N \cdot p_{\mathrm{f}}} \tag{107}$$

If a small coefficient of variation is required, for example 10%, then for probabilities of failure of for example $p_{\mathrm{f}} = 10^{-4}$, as many as $N = 10^6$ trials are needed (Schneider & Vrouwenvelder 2017). In order to increase the efficiency of the Monte Carlo method, importance sampling methods can be used, which will not be further discussed in this report.

Generation of Uniformly Distributed Random Numbers

The most common practical approach to generate distributed random numbers is to employ a 'pseudo' random number generator (PRNG), which is available on nearly all computer programs. The probability distribution for these numbers would be 'uniform' or 'rectangular'. The numbers generated are not truly random but are generated with a formula and are therefore termed 'pseudo'. The sequence of numbers generated is reproducible and repeats normally after a long cycle interval, but for most practical purposes, it is indistinguishable from a sequence of strictly true random numbers (Melchers & Beck 2018).

Generation of Random Variates

Basic variables rarely have a uniform distribution. A sample value for a basic variable with a given (nonuniform) distribution is called a 'random variate' and can be obtained by for example the 'inverse transform' method. The 'inverse transform' method is the most general method to use. The basic variable X_i has the cumulative distribution function $F_{\mathrm{X_i}}(x_\mathrm{i})$ and must by definition lie in the range [0,1] (Melchers & Beck 2018). This is shown in figure 39.

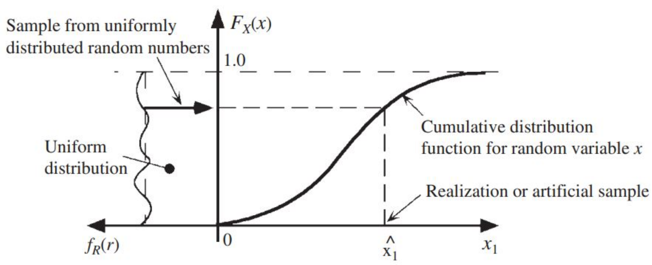


Figure 39: Inverse transform method for generation of random variates (Melchers & Beck 2018).

The inverse transform technique is to generate a uniformly distributed random number $r_\mathrm{i}(0 \leq r_\mathrm{i} \leq 1)$ and equate this to as $F_{\mathrm{X_i}}(x_\mathrm{i})$.

$$F_{\mathrm{X_i}}(x_\mathrm{i}) = r_\mathrm{i} \;\text{ or }\; x_\mathrm{i} = F_{\mathrm{X_i}}^{-1}(r_\mathrm{i}) \tag{108}$$

This uniquely fixes the sample value $x_\mathrm{i} = \hat{x}_\mathrm{i}$, provided that an analytic expression exists for the inverse $F_{\mathrm{X_i}}^{-1}(r_\mathrm{i})$ (which it does for the Weibull, exponential, Gumbell, rectangular, and more distributions) (Melchers & Beck 2018).

C Interview About Decision Making at NPRA

Name: Tor Anders Hagstrøm

Title: Senior Engineer in the Bridge and Ferry Quay department

Administration: Norwegian Public Roads Administration

Date of interview: 06.04.2022

Time of interview: 14:45-15:45

Questions

1. Are inspections of bridges planned and carried out in accordance with requirements/regulations? (inspection intervals, level of detail)
2. Are the inspection intervals ever reconsiderd/changed?
3. Are inspections ever put on hold?
4. Do newly constructed bridges have the same intervals for inspection, or is it different during the first years of the structure's lifetime?
5. After a bridge element is repaired, does the bridge follow the same inspection intervals right after the repair?
6. Is there the situation where you can't perform an inspection because of lack of funds, people, etc.?
7. Is the ROS-analysis performed for every bridge evaluated? Most of them?
8. Do you think the inspections are optimized after the ROS-analysis was introduced?
9. Are the inspections accurately registered in Brutus? (inspection carried out but registration is done some time later)
10. Are inspection results used in capacity assessments and safety assessments?
11. Is it common to put planned maintenance on hold?
12. Does NPRA carry out preventive maintenance?
13. If two methods of measurements/material testing are being considered in order to get better basis for determining the type, degree, extent, consequence, is the utility quantified, to justify the choice of one method over the other?
14. When performing a measurement, is there a rational on how many measurements should be done?
15. What criteria do you use to demonstrate sufficient safety of the bridge?

16. If NPRA does not follow their own guidelines, which are used to comply with legislations (made by the Road Supervisory Authority / NO: Vegtilsynet) that ensure high standard of safety on our national road network, do they (NPRA) have liability for damages, if a bridge would collapse?

17. Do you think there are enough funds for the coming years with ageing bridges?

18. If you only fix critical damage, won't there be a point in time where the other more minor damages have accumulated and you won't have the funds to fix all of them?

D Decision Theory Calculations

D.1 Priori Analysis

Table 19: Probabilites of failure and expected values of actions a_0 and a_1.

Year	$P'(\theta_0)$	$P(F \mid a_0)$	$P(F \mid a_1)$	$E'[u \mid a_0]$ [NOK]	$E'[u \mid a_1]$ [NOK]
5	0.10	10^{-6}	$1.009 \cdot 10^{-4}$	30,045	4,541
10	0.25	10^{-6}	$2.508 \cdot 10^{-4}$	30,045	11,284
15	0.50	10^{-6}	$5.005 \cdot 10^{-4}$	30,045	22,523
20	0.60	10^{-6}	$6.004 \cdot 10^{-4}$	30,045	27,018
25	0.75	10^{-6}	$7.503 \cdot 10^{-4}$	30,045	33,761
30	0.85	10^{-6}	$8.502 \cdot 10^{-4}$	30,045	38,257
35	0.90	10^{-6}	$9.001 \cdot 10^{-4}$	30,045	40,505

D.2 Posterior Analysis

Table 20: Visual inspection - given detection. Posterior probabilites of failure and expected values of different actions.

Year	$P''(\theta_0)$	$P''(\theta_1)$	$E[u \mid a_0]$ [NOK]	$E[u \mid a_1]$ [NOK]
5	0.26	0.74	30,045	11,836
10	0.52	0.48	30,045	23,248
15	0.76	0.24	30,045	34,296
20	0.83	0.17	30,045	37,249
25	0.91	0.09	30,045	40,759
30	0.95	0.05	30,045	42,650
35	0.97	0.03	30,045	43,491

Table 21: Sensor inspection - given detection. Posterior probabilites of failure and expected values of different actions.

Year	$P''(\theta_0)$	$P''(\theta_1)$	$E[u \mid a_0]$ [NOK]	$E[u \mid a_1]$ [NOK]
5	0.67	0.33	30,045	30,015
10	0.86	0.14	30,045	38,578
15	0.95	0.05	30,045	42,634
20	0.96	0.04	30,045	43,394
25	0.98	0.02	30,045	44,183
30	0.99	0.01	30,045	44,564
35	0.99	0.01	30,045	44,724

Table 22: Visual inspection - given no detection. Posterior probabilites of failure and expected values of different actions.

Year	$P''(\theta_0)$	$P''(\theta_1)$	$E[u \mid a_0]$ [NOK]	$E[u \mid a_1]$ [NOK]
5	0.03	0.97	30,045	1,339
10	0.08	0.92	30,045	3,715
15	0.21	0.79	30,045	9,509
20	0.29	0.71	30,045	12,889
25	0.44	0.56	30,045	20,025
30	0.60	0.40	30,045	27,098
35	0.71	0.29	30,045	31,778

Table 23: Sensor inspection - given no detection. Posterior probabilites of failure and expected values of different actions.

Year	$P''(\theta_0)$	$P''(\theta_1)$	$E[u \mid a_0]$ [NOK]	$E[u \mid a_1]$ [NOK]
5	0.01	0.99	30,045	565
10	0.03	0.97	30,045	1,569
15	0.10	0.90	30,045	4,326
20	0.14	0.86	30,045	6,175
25	0.24	0.76	30,045	10,834
30	0.37	0.63	30,045	16,841
35	0.49	0.51	30,045	21,915

D.3 Preposterior Analysis

D.3.1 Preposterior Expected Utilities

Table 24: Posterior expected utilities of visual inspection. Expected values are given in NOK.

Year	$E[u(e_1, z_0, a_0)] = E[u(e_1, z_1, a_0)]$	$E[u(e_1, z_0, a_1)]$	$E[u(e_1, z_1, a_1)]$
5	33,045	14,836	4,339
10	33,045	26,248	6,715
15	33,045	37,296	12,509
20	33,045	40,249	15,889
25	33,045	43,759	23,025
30	33,045	45,650	30,098
35	33,045	46,491	34,778

Table 25: Posterior expected utilities of sensor inspection. Expected values are given in NOK.

Year	$E[u(e_2, z_0, a_0)]$ $= E[u(e_2, z_1, a_0)]$	$E[u(e_2, z_0, a_1)]$	$E[u(e_2, z_1, a_1)]$
5	39,045	39,015	9,565
10	39,045	47,578	10,569
15	39,045	51,634	13,326
20	39,045	52,394	15,175
25	39,045	53,183	19,834
30	39,045	53,564	25,841
35	39,045	53,724	30,915

D.3.2 Outcome Probabilities, Expected Utilities for Each Inspection-Outcome Pairs, and Expected Utility

Table 26: The outcome probabilites, expected utilities for each inspection-outcome pairs, and the expected utility of inspection e_1.

Year	$P(z_0)$	$P(z_1)$	$u(e_1, z_0)$ [NOK]	$u(e_1, z_1)$ [NOK]	$E[u(e_1)]$ [NOK]
5	0.305	0.695	14,836	4,339	7,541
10	0.388	0.613	26,248	6,715	14,284
15	0.525	0.475	33,045	12,509	23,291
20	0.580	0.420	33,045	15,889	25,840
25	0.663	0.338	33,045	23,025	29,663
30	0.718	0.283	33,045	30,098	32,212
35	0.745	0.255	33,045	33,045	33,045

Table 27: The outcome probabilites, expected utilities for each inspection-outcome pairs, and the expected utility of inspection e_2.

Year	$P(z_0)$	$P(z_1)$	$u(e_2, z_0)$ [NOK]	$u(e_2, z_1)$ [NOK]	$E[u(e_2)]$ [NOK]
5	0.135	0.865	39,015	9,565	13,541
10	0.263	0.738	39,045	10,569	18,044
15	0.475	0.525	39,045	13,326	25,543
20	0.560	0.440	39,045	15,175	28,542
25	0.688	0.313	39,045	19,834	33,042
30	0.773	0.228	39,045	25,841	36,041
35	0.815	0.185	39,045	30,915	37,541

E Load-Carrying Capacity Classification by Norwegian Public Roads Administration

Load-carrying capacity classification of bridges is used to determine the maximum traffic loads for existing bridges. The regulations for the load-carrying capacity classifications consists of the two following manuals, published by NPRA:

V412: Load-Carrying Capacity of Bridges, Loads (NPRA 2021*c*) defines the traffic loads for which the bridges are to be checked for, as well as load factors and load combinations;

V413: Load-Carrying Capacity of Bridges, Materials (NPRA 2021*d*) states material strengths and material factors.

In addition, the manuals are used for the design of reinforcement and rebuilding of existing bridges. The guidelines have been updated in July 2021, where manual V412 replaces manual R412 (NPRA 2014) and manual V413 replaces an appendix that belongs to manual R412.

E.1 Manual V412 - Loads

Manual V412 (NPRA 2021*c*) defines the traffic loads for the use classes to be used when classifying existing bridges and ferry quays in the public road network. In addition, the manual specifies traffic loads for motorised equipment Sv 12/65, special transport according to Road groups A and B, special transport Sv 12/100, and provides the prerequisites for checking bridges for other special transports and one-off transports. The traffic loads are based on the axle load and total weight provisions given in the Ministry of Transport's Regulations on the use of vehicles. The manual V412 uses the partial factor method for design checks. In this section, the guidelines described in V412 will be presented.

E.1.1 Definition of Loads

A load is defined as any kind of impact that causes stresses and strains in a structure.

Terminology for Load-Carrying Capacity Classification of Bridges

Traffic Loads on existing bridges are all traffic loads that are allowed in the public sector road networks, such as use classes, motorised equipment (Sv 12/65), special transports (road groups, Sv 12/100 and others) and one-off transports;

Use Class is the traffic load that is allowed to drive freely without dispensation. Use class is indicated by the abbreviation 'Bk' followed by the maximum permissible axle load/total weight. Bk 10/50 denotes, for example, use class with axle load of 10 tonnes (100 kN) and maximum permissible total weight of 50 tonnes (500 kN);

Motorised Equipment Sv 12/65 is the road network for motorised equipment. It is used by for example mobile cranes, concrete pump trucks, and lifts. The maximum permissible axle load is 12 tonnes (120 kN) and the maximum permissible total weight is 65 tonnes (650 kN). A general dispensation is granted without a time limit for driving in road networks for motorised equipment. This means that one can drive freely together with ordinary traffic;

Special Transport is the transport of indivisible goods (such as the transport of machines and building modules) that place a greater load on the road network than permitted classes of use, so that a dispensation must be obtained before driving;

Special Transport, Road Group provides a road network for special transports based on permitted classes up to Bk 10/50. Road group allows for two types of special transports:

1. Special transport without time limit. Requires a general dispensation. Can be is driven without supervision. It is driven unaccompanied together with other traffic;
2. Special transport with time limit. Requires dispensation in each individual case. There will then be restrictions for crossing bridges and the transport is followed/supervised.

Special Transport, Road Groups three different road groups are used:

1. *Road group A* bridges that have two or more lanes, newer bridges with one lane when they are designed for regulatory load SVV 1969 or newer. For older, one lane bridges classified as Road group A, the classification calculation must demonstrate satisfactory load-carrying capacity. Bridges classified as Bk 10/50 A can also be used by motorised equipment with 2- and 3-axles with axle loads up to 12 tonnes (120 kN) and total weight up to 36 tonnes (360 kN);
2. *Road Group B* all bridges that are not classified for Road Group A can normally be classified for Road Group B. The exception are bridges which for some reason have vulnerable load-carrying capacity and are therefore classified for Road Group NOT;
3. *Road Group NOT* used in special cases for bridges with vulnerable load-carrying capacity. A dispensation for the special transport on bridges that are classified as Road group NOT are not given.

Special Transport, Sv 12/100 provides a road network for special transports with axle loads up to 12 tonnes (120 kN) and a total weight of up to

100 tonnes (1000 kN). Only a time-limited dispensation is granted and the transport must be supervised if the bridge is not cleared for free travel together with other traffic. Motorised equipment (Sv 12/65) have a permission to drive with a time-limited dispensation in the road network that is open for Sv 12/100;

Special Transport, Other applies to all special transports where the section to be driven is not covered by the road network for Road groups or Sv 12/100. It is normally driven with a supervision;

One-Off Transport is transport with great social significance such as transport of transformers and equipment to the existing power supply. It is normally driven with supervision;

Axle Load is the load from all wheels on one axle;

Bogie Load load from axle combinations with two axles where the axle distance is less than 1.80 m;

Triple Axle Load load from axle combinations with three axles where the mutual distance between each axle is less than 1.80 m;

Total Weight load from the entire vehicle or truck;

UF transport that drives freely with other traffic;

MF transport with supervision to ensure that restrictions for bridge crossing complied with and at the same time handle other traffic flow over the bridges.

Classification of Loads

Loads that are to be used as a basis for calculating design load effects are designated as characteristic loads. Characteristic traffic loads are loads that are allowed in the road network. The magnitude of the characteristic load depends on whether it occurs:

- In temporary phases such as during construction, installation, and removal;
- In normal use, for example ordinary traffic loads;
- In case of accidental load or abnormal traffic or natural load;
- In a damaged condition.

In the manual, the characteristic loads are divided into:

Permanent Loads loads that can be considered constant within the time period considered. They can for example include the weight of the structure (self-weight), weight of the wear layer of roads, and the pressure and weight of soil;

Variable Loads loads that vary in time and can for example include traffic loads, natural loads, and short term operations;

Deformation Loads loads that are associated with deformations such as deformation due to construction or mounting, or the construction material properties, such as shrinkage, creep, and relaxation. Deformation loads are often time-dependent and the characteristic load is defined as the largest expected value within the time period considered;

Accidental Loads loads caused by accidental occurrences or abnormal events such as loads caused by landslides and collision loads from vehicles.

With two or more loads that are dependent on time and location, or loads that often occur with their maximum value at the same time, the loads are considered as a combination of loads.

E.1.2 Traffic Loads

General

Traffic Loads on Existing Bridges

Traffic load is the load in both the vertical and horizontal direction on the roadway, shoulder of the roadway, walkway, bicycle lane, and the central reservation from both pedestrians and all vehicles than can load the structure. Traffic loads are divided into the following subchapters in manual V412:

3.2 Ordinary use classes (Bk 6/28, Bk 8/32, Bk T8/40, Bk T8/50, Bk 10/50);

3.3 Special use classes (Bk 10/60, Bk 10/74);

3.4 Motorised equipment (Sv 12/65);

3.5 Special transports, Road groups (A, B, NOT);

3.6 Special transports (Sv 12/100);

3.7 Special transports, Other (Special transports not covered by Road group or Sv 12/100);

3.8 One-off transports.

The traffic load is placed on the bridge in the most unfavourable position in the longitudinal and transverse direction.

Basis for Equivalent Loads

The traffic loads provided in this manual are equivalent loads. For pedestrian/bicycle lanes and pedestrian/bicycle bridges, the equivalent loads are based on load regulations SVV 1995 (Internordisk) and SVV 2010 (Eurocode).

Dynamic Addition

The manual includes dynamic addition in the traffic loads. For special transports and Road groups it applies in particular that bridge crossing with supervision (MF) is assumed to be slow and central on the bridge. Traffic loads for Road group with supervision therefore do not have dynamic additions. The dynamic surcharge incorporated in the equivalent loads is 40% on the heaviest axle.

Design Shear Force

The equivalent loads for load-carrying capacity classification are based on moment comparisons with traffic loads. This means that the total weights for the equivalent loads are somewhat larger than real total weights, which in turn means that design shear can be somewhat conservative. Therefore it is possible to reduce this somewhat if there is high utilisation in relation to capacity.

How Loads are Determined for Different Traffic Loads

In the manual, guidelines of how loads are determined for each of the types of traffic loads are provided. In this section, only the guidelines for ordinary use classes will be described. This is demonstrated below.

Ordinary Use Classes

General

Ordinary use classes cover normal transport with a total weight of up to 50 tonnes (500 kN).

Vertical Loads

When classifying load-carrying capacity of bridges, ordinary use classes shown in fig. 40 are used.

Tabell 3-1: Ordinære bruksklasser

Bruksklasse	Aksellast	Totalvekt
Bk 10/50	115 kN	500 kN
Bk T8/50	80 kN	500 kN
Bk T8/40	80 kN	400 kN
Bk 8/32	80 kN	320 kN
Bk 6/28	60 kN	280 kN

Figure 40: Ordinary use classes (NPRA 2021*c*).

Bk T8/40 is a variant of Bk 8/32. It has the same maximum axle load as Bk 8/32, but triple bogie load, vehicle load, and the truck load are higher. The same applies to Bk T8/50, where the truck load has also been increased to 50 tonnes (500 kN).

The equivalent loads for each of the use classes consist of a bogie load, vehicle load, and truck load. Vehicle load and truck load have been converted into a set of axles with equal wheelbase. For truck load, an evenly distributed additional load of 6 kN/m (3 kN/m^2) is included, which is an average load for a reasonable mix of light and heavy, empty and fully loaded vehicles.

The vertical loads for the ordinary use classes are given in fig. 41 below. The abbreviations in the figure have the following meaning: A = Axle load, a= Wheelbase.

Figur 3-2 Ekvivalentlaster for ordinære bruksklasser

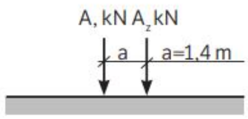
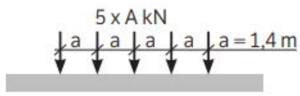
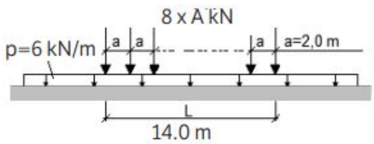
Lasttype	Lastfordeling	Ordinære bruksklasser					
			Bk 10/50	Bk T8/50	Bk T8/40	Bk 8/32	Bk 6/28
Boggilast	 A, kN A, kN; a; a=1,4 m	A1	165	125	125	125	100
		A2	120	90	90	55	35
Kjøretøy-last	 5 x A kN; a a a a a=1,4 m	A	80	68	68	58	45
Vogntog-last	 8 x A kN; p=6 kN/m; a a a a=2,0 m; L; 14.0 m	A	60	55	47	38	30

Figure 41: The vertical loads for the ordinary use classes (NPRA 2021*c*).

The Size and Location of the Load Field in the Transverse Direction

The vertical loads of the use classes are placed on the bridge in the most unfavourable position in the transverse direction, within the available driving space. The driving space is the minimum horizontal width of the distance between the road curbs, distance between the road curb and tall curb/railing, and distance between two tall curbs or railings.

The width of the load field with heavy vehicles/trucks is 3,0 m. The evenly distributed load of 6 kN/m is calculated to have a load field of 2,0 m, seen in fig. 42. The symbols in the figure are F = driving space, T = width requirement for heavy traffic load (3,0 m), and t = width requirement for light traffic load (2,0 m).

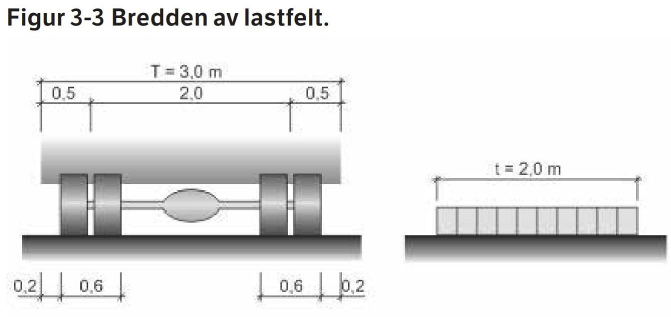


Figure 42: The width of a load field (NPRA 2021*c*).

The contact surface of the wheel load (the load of a vehicle that is carried by a single wheel) is a rectangle with sides 0,2 m in the longitudinal direction and 0,6 m in the transverse direction. The width of the vehicle is considered to be 2,6 m. In addition, a free space of 0,2 m outside the vehicle at a height of 0,4 m above the roadway is required. Figures 43 and 44 show examples of different positions of vehicles with different types of railings and curbs.

In order for the curbs to be the deciding factor when determining the driving space, the height difference of the top edge of the curb and the top of the road surface should be at least 90 mm. This is shown in fig. 44. If this difference is lower, the driving space should include the distance all the way to the tall curb/railing.

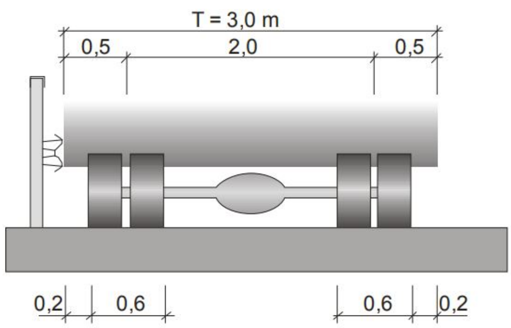


Figure 43: Position of a vehicle sideways next to steel railings without curbs (NPRA 2021*c*).

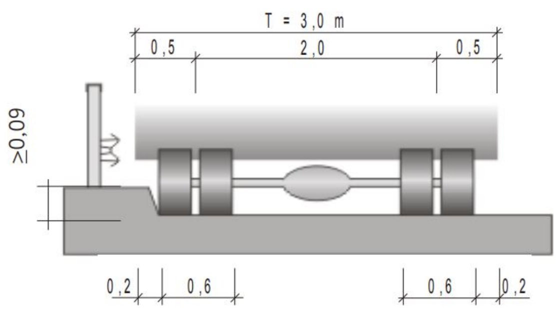


Figure 44: Position of a vehicle sideways next to a curb (NPRA 2021*c*).

Trucks and evenly distributed traffic loads in the same load field are assumed to have the same eccentricity.

A maximum of two load fields are loaded with bogie loads, vehicle loads, or truck loads. The trucks have in addition an evenly distributed load. Other load fields only have an evenly distributed load of 6 kN/m. The load fields are placed in the most unfavourable position in the whole area which is accessible for driving traffic, including shoulders and other surfaces in the roadway plan. Outside of the load fields there are no traffic loads.

The manual shows examples of how an arrangement is made of the number of load fields for driving spaces $\geq$ 2,6 m and with varying curbs/edges. Bridges with driving space $<$ 2.6 m are not normally checked for use classes. In some cases, it must be assessed whether there is a need for an additional checks for an unusual traffic load that are caused by two heavy load tracks. This load is treated as an accidental load. For two-lane bridges, horizontal curvature on the bridge, or on an adjacent road towards the bridge, can mean that there can normally be only one heavy load track on the bridge at a time. In such cases, the bridge can be calculated as single lane if it is additionally checked for an unusual traffic load consisting of two heavy load tracks. This load is also treated as an accidental load.

Horizontal Loads

The horizontal loads include brake load, side load, and centrifugal load. These loads can not act alone, only at the same time as the associated vertical loads shown in fig. 41.

Brake Load (B)

The effect of vehicle braking and acceleration in a load field is calculated based on a horizontal load B1 at bridge length $\leq$ 10 m and B2 at bridge length $\geq$ 40 m. The bridge length is the total length of the bridge part or parts which can simultaneously transfer brake load to the structural part to be inspected. The brake load is assumed to act in the longitudinal direction of the bridge at the height of the road way, and can be assumed to be evenly distributed over the entire width of the road way.

The brake load varies with the different use classes, as shown in fig. 45. For bridge lengths between 10 and 40 m, linear interpolation is used. Figure 46 shows the brake load graphically.

Tabell 3-2 Bremselaster

Brulengde	Bremselast (kN)			
	Bk 10/50 Bk T8/50	Bk T8/40	Bk 8/32	Bk 6/28
≤ 10 m (B1)	150	120	100	90
≥ 40 m (B2)	300	240	190	170

Figure 45: Brake loads for different use classes (NPRA 2021*c*).

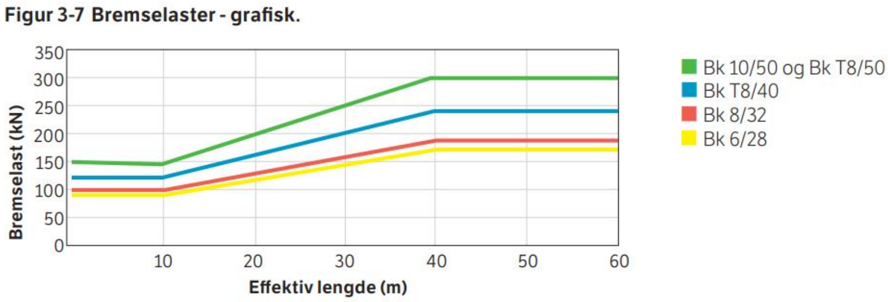


Figure 46: Brake loads shown graphically (NPRA 2021*c*).

If the bridge has two or more load lanes in the same direction of travel, the brake load is equal to 1,5B.

Side Load (S)

The effect of a skewed or asymmetrical braking of vehicles, side impacts, etc., is calculated as arbitrarily placed horizontal load S = 25% of the braking load. It acts simultaneously with the brake load and the vertical traffic load. Side loads are assumed to act perpendicular to the longitudinal direction of the bridge and at the height of the road way.

Centrifugal Load (SC)

Centrifugal load works simultaneously with vertical traffic load, but not together with brake load and side load. The formula for centrifugal load is:

$$S_c = v^2 \cdot V/(127 \cdot R) \leq 0,2 \cdot V[\text{kN or kN/m}] \tag{109}$$

where

v = maximum speed [km/h];

R = radius of horizontal curve [m];

V = vertical load in kN for the axle loads and in kN/m for evenly distributed load.

The maximum speed is usually set to 70 km/h. In densely populated areas where the speed is lower, the maximum speed can be reduced but not to lower than 50 km/h. If R $\geq$ 1500 m the centrifugal load does not need to be taken into account.

Fatigue Load

Fatigue load is assessed separately in each individual case. It depends on on age, traffic history, and how vulnerable the construction is believed to be for such loads.

Simultaneous Load on the Pedestrian and Bicycle Lane

Traffic load on the pedestrian and bicycle lane on road bridges at the same time as traffic load in the road way depends on how the lanes are seperated and on the width of the pedestrian and bicycle lane. The manual provides information on how to calculate the load on pedestrian and bicycle lanes in cases where pedestrian and bicycle lanes are separated from the road way by railings, when pedestrian and bicycle lane is separated from the road way with a curb, and when the pedestrian and bicycle path levels with the road way.

Traffic Loads on Bridge Embankments

The manual describes how support structures for the bridge, such as abutments and retaining walls should be loaded with traffic loads and other payloads. It is also shown how the earth/soil pressure from the load is calculated. It is described how support structures such as abutments and retaining walls for a bridge, both a road bridge and pedestrian and bicycle bridges, should be loaded with traffic load and other payload. It is also shown how the earth/soil pressure from the load is calculated. The manual takes into account bridges on pedestrian and bicycle paths that are also used as road access to homes or the like. They are considered as road bridges and are classified accordingly.

E.1.3 Permanent Loads

Dead load

When classifying the load-carrying capacity of bridges, the manual provides the following dead loads that should be used:

- Steel = 77 kN/m^3;
- Aluminium = 27 kN/m^3;
- Reinforced concrete = 25 kN/m^3;
- Asphalt = 25 kN/m^3;
- Oil gravel = 22 kN/m^3;
- Stone = 25 kN/m^3;
- Gravel = 18 kN/m^3;
- Lightweight expanded clay aggregate (LECA), stabilized = 8 kN/m^3;
- Wooden deck without moisture insulation = 8 kN/m^3;
- Wood, structural timber = 5 kN/m^3;
- Steel railing = 0.5 kN/m.

The permitted thickness of the pavement should be calculated for Use class, Road groups, Sv 12/65, and Sv 12/100. Permitted pavement thickness for the bridge will be the minimum thickness for those of the traffic loads that become the current load-carrying capacity classification for the bridge. The actual pavement thickness must be checked against permissible pavement thickness, for example in connection with inspection.

Water Pressure and Earth Pressure

Variable water pressure loads are due to variations in water- or groundwater level. Characteristic values are determined on the basis of the highest and lowest observed water level. For groundwater level the limits should be assessed separately. If efficient and lasting drainage is provided, this can be taken into account when determining variable water pressure load. Ground pressure is calculated in accordance with manual V220.

E.1.4 Other Loads

Natural Loads

Snow Load

Snow loads are not considered to occur at the same time as traffic loads on road bridges and ferry quays. If the construction part can be used as storage space for snow, or is not expected to be cleared of snow, the load must be assessed further.

Wind Load, Hydrodynamic Load, Ice Load, Temperature Load, and Earthquake Load

When checking existing bridges in load combination b, wind loads from the original calculations can be used. If it is relevant to check for hydrodynamic loads, ice loads and temperature loads, the regulations that were used when designing the bridge can also be used. Earthquake loads are an abnormal natural load and most older bridges are not designed for such loads. Therefore, when classifying load-carrying capacity of bridges, it is normally not necessary to check for earthquake loads.

Deformation Loads

Deformation loads from tensioning, shrinkage, creep, and relaxation should be taken into account when determining load-carrying capacity classification of bridges. Any settlements and their future developments should be taken into account.

Accidental Loads

Usually, it is not necessary to check bridges for accidental loads in connection with load-carrying capacity classification of bridges.

E.1.5 Calculation of Load-Carrying Capacity Classification

Classification Methods

The purpose of the calculations is to demonstrate that the design loads do not exceed the design resistance in the structure. Usually, elastic methods should be used to calculate the design load and resistance. Alternative classification methods, such as classification by test load, use of probabilistic methods, etc., are assessed in consultation with NPRA before implementation.

Control of Boundary Conditions

When classifying load-carrying capacity of bridges, it may be relevant to carry out inspections in the ultimate limit state, serviceability limit state, accidental limit state, and in some cases the fatigue limit state.

The design load effect is determined by combining the effect of the characteristic load multiplied by a load factor. The load factors are specified in chapter 9 in the manual. The design resistance is determined based on the characteristic resistance and material factors, which are specified in construction standards and in manual V413. Design against fatigue shall be based on either S-N curves or fracture mechanical crack growth analyses. Design load and resistance can be calculated using deterministic calculation models. Normal uncertainties in the calculation models are assumed to be covered by the partial factors. If the calculation models are particularly uncertain, models that are reasonably conservative for the critical parts of the construction should be chosen.

Inspections, Field Measures, Model Testing, Etc.

In case of uncertainty related to the condition of the structure or its execution, the need for inspection must be considered. If loads, load effects, or resistance have great uncertainty or can not be determined with reasonable accuracy, field measurements and/or model tests should be performed.

E.1.6 Design Load Effects

Calculation of Load Effects

The load effects should be determined using recognised methods that take into account the variation of the loads, both in time and space, the response of the structure, the relevant natural and ground conditions, as well as the boundary condition to be checked. Simplified methods can be used if it is sufficiently documented that they provide results to the safe side. The effect of the displacements of the structure should be taken into account when calculating forces and moments in constructions and construction parts. The buckling length of structures shall be determined in accordance with their end conditions.

Design Load Combinations

Ultimate Limit State

For use classes, a check must be made as a minimum in the ultimate limit state. It must be considered if checks should be made in the other limit states. The load combinations for motorised equipment, special transports and one-off transports are usually only checked in the ultimate limit state. They are checked for two sets of load combinations, which are given in fig. 47.

Tabell 9-1 Lastfaktorer for bruddgrensetilstanden

Lastgruppe Kombinasjon	Permanente laster, P		Deformasjons-laster, D	Variable laster Q
	Jordtrykk, J	Egenlast/Andre		
a	1,0	1,15 (1) (2)	γ_D	$\gamma_1 \cdot Q_1$
b	1,0	1,0	1,0	$\gamma_2 \cdot Q_1 + 0,8 \cdot \Sigma Q_n$

(1) Ved kontroll for engangstransporter settes lastfaktor for egenlast og andre permanente laster til 1,1.
(2) Lastfaktor for permanente laster settes lik 1,0, dersom dette er ugunstigere.

Figure 47: Load factors for the ultimate limit state (NPRA 2021*c*).

Where Q_1 is the characteristic value of the variable load that is most unfavourable for the load effect considered and Q_n is the characteristic value for other variable loads that are unfavourable for the load effect. The load factors γ_D, γ_1, and γ_2 vary depending on different types of traffic loads and different load configurations. They are provided in chapter 9 in manual V412.

Servicability Limit State

If a check needs to be done in the serviceability limit state, the two sets of load combinations shown in fig. 48 are checked. The combination factors are provided in chapter 9 of the manual.

Tabell 9-2 Lastfaktorer for bruksgrensetilstanden

Kombinasjon	Permanente laster P	Deformasjons-laster D	Variable laster, Q		
			Trafikklast T	Naturlast E	Ballast etc. L
a	1,0	1,0	$Q_1 + 0,7 \cdot \Sigma Q_n$		1,0
b	1,0	1,0	$\Psi_1 \cdot Q_1 + 0,7 \cdot \Sigma \Psi_1 \cdot Q_n$		

Figure 48: Load factors for the serviceability limit state (NPRA 2021*c*).

Combination a is assumed to represent the largest expected load condition during the lifetime of the structure and is used for checking of bearing, joint displacements, etc.

Combination b is assumed to represent a load condition which will not be exceeded 100 times during the service life of the structure, and is used for crack width control of concrete structures and for control of typical deformations and displacements.

Crack width check is not normally performed for load-carrying capacity classification of bridges. In chapter 9, the manual specifies for what elements and conditions this should be done. The manual also provides special requirements for suspension bridges that need to be fulfilled.

Accidental Limit State

For use classes, where relevant, a check of the accident limit state for combination a with a load factor, should be done. The load factors for the accidental limit state are shown in fig. 49.

Combination a the structure is exposed to an unusual action such as accidental load or unusual traffic and natural load.

If the bridge is considered to be at a high risk of being affected by accidental loads, unusual traffic loads, or natural load, a check should be done for the load-carrying capacity classification of bridges in accordance with manual V412. The load factor is provided in chapter 9 of the manual.

Tabell 9-4 Lastfaktorer for ulykkesgrensetilstanden

Kombinasjoner	Permanentlast, P	Deform. last, D	Variable laster, Q			Ulykkeslast, A
			Trafikk-last, T	Natur-last, E	Ballast etc., L	
a	1,0	γ_D	0	0	1,0	1,0

Figure 49: Combination factors (NPRA 2021*c*).

Fatigue Limit State

In special cases where it is necessary to perform fatigue checks to determine the load-carrying capacity classification of bridges, fatigue checks are carried out in accordance with current Norwegian standards and manuals.

E.2 Manual V413 - Materials

Manual V413 (NPRA 2021*d*) specifies values for material strength and material factors for concrete structures, steel structures, timber structures, and stone arch bridges. In this section, only the part about concrete structures will be presented.

Since 1973-74, the partial safety factor method has been used in Norwegian design standards. The material factors and material strengths provided in manual V413 are used for the load-carrying capacity classification and the design of reinforcement and rebuilding of existing bridges. The design load is checked against the design capacities calculated according to Eurocode standards. For concrete bridges designed before the Eurocode standards were developed and put in use, NS 3473, 6th edition 2003 can be used for calculation of design shear capacity.

E.2.1 Concrete Structures

Capacity control is performed in accordance with NS-EN 1992 with the subsequent material factors and material strength.

Material Factor - Concrete Structures

The material factor γ_c for concrete and γ_s for reinforcement steel are given in fig. 50. The values vary depending on when the concrete was reinforced.

Tabell 2.1.1 Materialfaktorer for betong og armeringsstål

Materiale	Materialfaktorer γ_C og γ_S			
	Bruddgrense-tilstand	Bruksgrense-tilstand	Ulykkesgrense-tilstand	Utmattings-grensetilstand
Armert betong, γ_C	1,50	1,0	1,20	1,50
Armering før 1920, γ_S	1,50[1]	1,0	1,30	1,30
Armering 1920-1955, γ_S	1,30[2]	1,0	1,20	1,20
Armering 1955-2010, γ_S	1,25	1,0	1,10	1,15
Armering etter 2010, γ_S	1,15	1,0	1,00	1,15

(1) For brudekker uten tegn til armeringskorrosjon av betydning for bæreevne i kritiske snitt, kan $\gamma_s = 1{,}30$ benyttes.
(2) For bruer uten tegn til armeringskorrosjon av betydning for bæreevne i kritiske snitt, kan $\gamma_s = 1{,}25$ benyttes.

Figure 50: Material factors for concrete and reinforcement steel (NPRA 2021*d*).

Design Material Strength - Concrete Structures

The design compressive strength of concrete is:

$$f_{\mathrm{cd}} = \alpha_{\mathrm{cc}} \cdot f_{\mathrm{ck}} / \gamma_{\mathrm{c}} \tag{110}$$

where

$\gamma_c = 0,85$;

f_{ck} = the characteristic cylinder compressive strength of the concrete after 28 days, see fig. 51;

γ_c= material factor, see fig. 50.

Figure 51 gives an overview of the characteristic cylinder compressive strength to be used for load-carrying classification depending on the year of construction of the bridge and the strength class of the bridge. The approximate connection between the concrete qualities in the old obsolete Norwegian standards NS 427 and NS427A as well as the strength classes in NS 3473 and NS-EN 1992 are also shown in the table. Values other than those specified in fig. 51 can be used if they are detected by drilling cores and pressure tests.

Tabell 2.1.2 Betongens karakteristiske trykkfasthet, f_{ck}

Byggeår	NS 427 (av 1939)	NS 427A (av 1962)	NS 3473 (1973-2003)	NS 3473 (2003-2010)		NS-EN 1992-1-1 (NA 3.1.2)	
	Betong-kvalitet	Betong-kvalitet	Fasthets-klasse	Fasthets-klasse	f_{cn} (N/mm^2)	Fasthets-klasse	f_{ck} (N/mm^2)
Før 1920	C-betong	B 200	C 15	B 10	11,2	B 12	12
1920-1945	B-betong	B 250	C 20	B 16	14,0	B 16	16
Etter 1945	A-betong	B 300	C 25	B 20	16,8	B 20	20
		B 350	C 30	B 25	20,3	B 25	25
		B 400	C 35	B 28	22,4	B 28	28
		B 450	C 40	B 32	25,2	B 32	32
			C 45	B 35	27,3	B 35	35
		B 600	C 55	B 45	34,3	B 45	45

For bruer som er bygd etter 1945 skal det ikke benyttes høyere fasthetsklasse enn C25/B20 dersom ikke annet er gitt på originaltegningene eller fremgår av annen dokumentasjon/ regelverk.

Figure 51: Overview of the characteristic cylinder compressive strength (NPRA 2021*d*).

For bridges built after 1945, a higher strength class than C25/B20 should not be used unless otherwise stated on the original drawings or in other documentation/regulations.

Design Reinforcement Strength (f_{yd})

The design yield strength of the reinforcing steel is calculated according to NS-EN 1992:

$$f_{yd} = f_{yk}/\gamma_s \tag{111}$$

where

f_{yk} = the characteristic yield strength of the reinforcement, see fig. 52;

γ_s = material factor, see fig. 50.

In bridges built in accordance with regulatory load SVV 1958 and newer, ribbed steel is usually used. Plain steel was mainly used prior to this, but there may also be bridges with ribbed steel or a combination of ribbed steel and plain steel. This must then appear from drawings, bending lists (a document consisting of geometric and material details for the reinforcement), calculations, or other documentation. It may also be detected by dismantling. If the reinforcement quality for ribbed steel is unknown, Ks40 is used. The characteristic yield strength of the reinforcement, f_{yk}, is shown in fig. 52.

Tabell 2.1.3 Armeringens karakteristiske flytegrense, f_{yk}

Armeringstype	Armeringskvalitet	Diameter (mm)	f_{yk} (N/mm²)
Glattstål	St. 37	8-32	230
Kamstål	Ks 40 og Ks 40 S	8-20 25-32	400 380
	Ks 50 og Ks 50 S	8-16 20-32	500 480
	Ks 60 og Ks 60 S	8-16	600
	K 400 S og K400 TS	8-32	400
	K500 S og K500 TS	8-32	500
	K500 TE	8-32	500
	B500C	8-32	500

Figure 52: Values for the characteristic yield strength of the reinforcement (NPRA 2021*d*).

Shear Capacity

Figure 53 gives values for the concrete's structural tensile strength, (f_{tn}), used when NS3473 is added as a basis. For a bridge designed according to NS 3473 6th edition 2003 and older Norwegian concrete standards, control of shear capacity can be performed in accordance with NS 3473, 6th edition 2003.

Tabell 2.1.4 Betongens konstruksjonsfasthet for strekk, f_{tn}

Fasthetsklasse	f_{tn} (N/mm²)
B 12	0,90
B 16	1,20
B 20	1,40
B 25	1,60
B 35	2,00
B 45	2,30

Figure 53: Values for the concrete's structural tensile strength, f_{tn} (NPRA 2021*d*).

If sufficient shear capacity is not demonstrated according to Eurocodes, a check shall also be made (or only) based on NS 3473.

F Service Life Model Code

F.1 Carbonation Depth

```
close all; clear all; clc;
% parameters
mu_a = 35; SD_a = 10;
mu_RH_real = 70; SD_RH_real = 11.2;
mu_R_ACC0 = 4225.8; SD_R_ACC0 = 1646.2;
mu_epsilon_t = 315.5; SD_epsilon_t = 48;
mu_C_s = 0.00082; SD_C_s = 0.0001;
mu_b_w = 0.446; SD_b_w = 0.163;

% Constants
RH_ref = 65; g_e = 2.5; f_e = 5.0; b_c = -0.567; t_c = 3.0;
k_t = 1.25; ToW = 0.2; p_SR = 0; t_0 = 0.0767;
% Finding k_c and k_e
k_c = (t_c/7)^b_c;
k_e = ((1-(mu_RH_real/100)^f_e)/(1-(RH_ref/100)^f_e))^g_e;
%% Carbonation depth
t_SL = [0:1:50];
t = [];

for i = 1:length(t_SL)
    t(i) = t_SL(i);
    W(i) = (t_0/t(i))^(((p_SR*ToW)^mu_b_w)/2);
    x_c(i) = sqrt(2*k_e*k_c*(k_t*mu_R_ACC0 + mu_epsilon_t)*mu_C_s)*sqrt(t(i))*W(i);
end

figure(1)
plot(t,x_c, 'r', 'Linewidth', 1)
ylabel('Carbonated depth [mm]')
xlabel('Time [years]')
title('Predicted Carbonation Development')
xlim([0, 50])
grid on
saveas(gcf, 'Carbonation_Depth', 'epsc')
```

F.2 Prediction Model Function

```
function [p_f, beta] = carbonation(mu_R_ACC0, SD_R_ACC0, t_SL, n)
```

```
% parameters
mu_a = 35; SD_a = 10;
mu_RH_real = 70; SD_RH_real = 11.2;
mu_epsilon_t = 315.5; SD_epsilon_t = 48;
mu_C_s = 0.00082; SD_C_s = 0.0001;
mu_b_w = 0.446; SD_b_w = 0.163;
mu_b_c = -0.567; SD_b_c = 0.024;
mu_k_t = 1.25; SD_k_t = 0.35;

% Constants
RH_ref = 65; g_e = 2.5; f_e = 5.0;
t_c = 3.0; ToW = 0.2; p_SR = 0;
t_0 = 0.0767;

%% Probability of Failure Calculations
t = [];
p_f = [];
k_e = [];

for i = 1:length(t_SL)
    t = t_SL(i);
    count =0;
    for j = 1:n
        a = normrnd(mu_a,SD_a); %resistance
        % Load
        RH_real = normrnd(mu_RH_real, SD_RH_real);
        R_ACC0 = normrnd(mu_R_ACC0, SD_R_ACC0);
        epsilon_t = normrnd(mu_epsilon_t, SD_epsilon_t);
        C_s = normrnd(mu_C_s, SD_C_s);
        b_w = normrnd(mu_b_w, SD_b_w);
        b_c = normrnd(mu_b_c, SD_b_c);
        k_t = normrnd(mu_k_t, SD_k_t);

        k_e = ((1-(RH_real/100)^f_e)/(1-(RH_ref/100)^f_e)
           )^g_e;
        k_c = (t_c/7)^b_c;
        W = (t_0/t)^(((p_SR*ToW)^b_w)/2);
        x_c = sqrt(2*k_e*k_c*(k_t*R_ACC0 + epsilon_t)*C_s
           )*sqrt(t)*W;

        G = a-x_c;
        if G < 0
            count = count + 1;
        end
    end
        p_f(i) = ((count/n)*100);
        beta(i) = -norminv(p_f(i)/100)
```

```
end
end
```