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Levetidsanalyse av kantdragere for tre bruer

FoU-programmet Bedre bruvedlikehold
2017-2021

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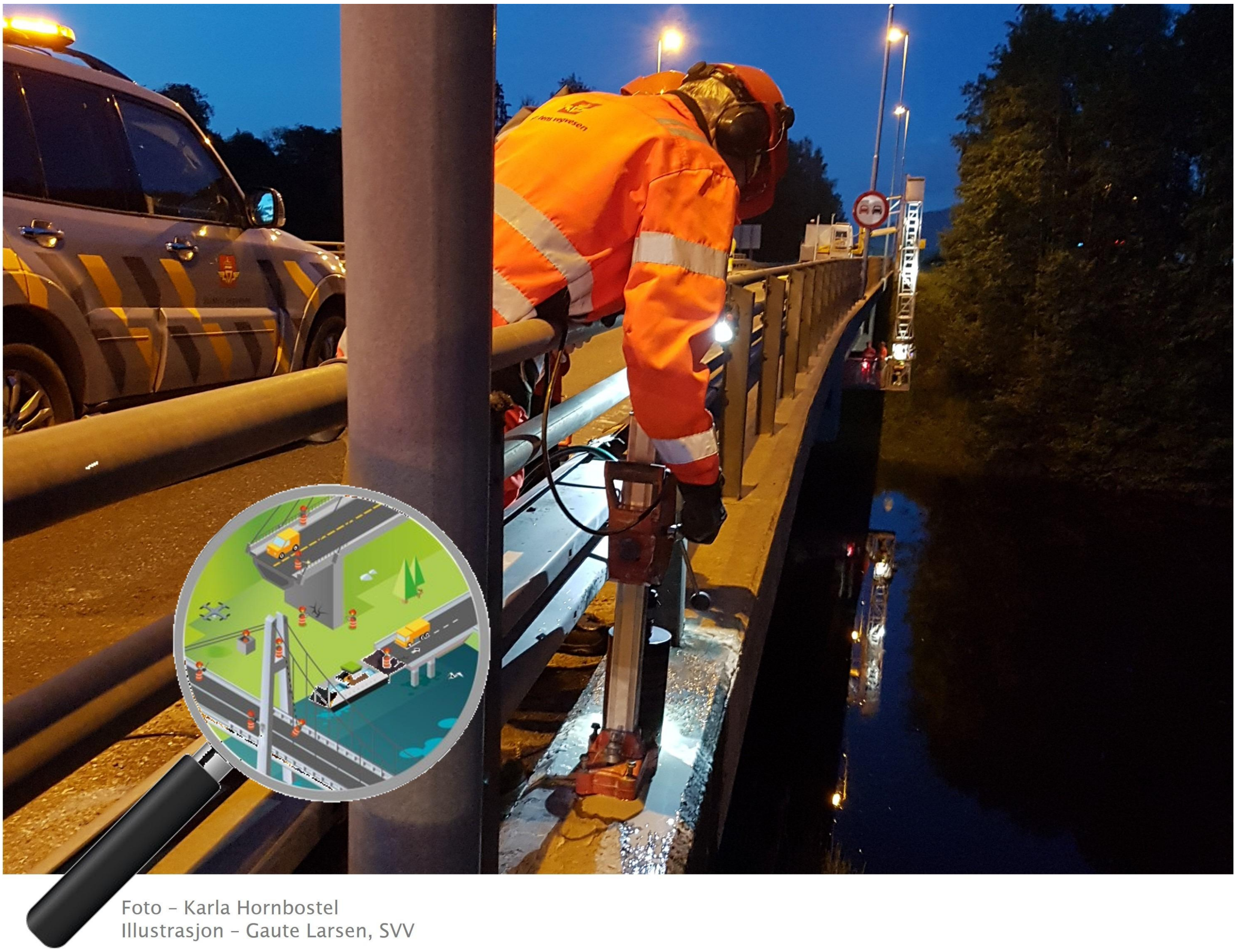

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NPRA reports
Norwegian Public Roads Administration

Tittel
Levetidsanalyse av kantdragere for tre bruer

Undertittel
FoU-programmet Bedre bruvedlikehold 2017-2021

Forfatter
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Emneord
Levetidsanalyse, armeringskorrosjon, klorider

Sammendrag
For planlegging av kostnadseffektivt vedlikehold av infrastruktur er det viktig å vurdere tilstanden til konstruksjoner og å estimere gunstig tidspunkt, omfang og type reparasjon som er nødvendig for å drifte konstruksjoner på en sikker måte. I denne studien presenteres en ny tilnærming som kombinerer data fra tilstandsvurdering av armerte betongkonstruksjoner (f.eks. kloridprofiler), prediktiv modellering (fremtidig kloridinntrenging) og eksperimentelt bestemte kloridterskelverdier (Ccrit) på kjerner hentet fra konstruksjonene med en metode nylig utviklet ved ETH Zürich, Sveits. Denne tilnærmingen brukes på tre bruer i Norge.

Antall sider 24

Title
Modelling Life Cycle Costs for edge beams of three Norwegian bridges

Subtitle
R&D program Better Bridge Maintenance 2017-2021

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Summary
For the planning of cost-effective infrastructure maintenance, it is important to assess the condition of structures and to estimate the optimum time, the extent and the type of repair needed to continue operating the structures safely. In this study, a novel approach is presented, which combines data from condition assessment of reinforced concrete structures (e.g. chloride profiles), predictive modelling (future chloride ingress) and experimentally determined chloride threshold values (Ccrit) on cores retrieved from the structures with a method recently developed at ETH Zurich, Switzerland. This approach is applied to three bridges in Norwa

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Final Report

Modelling Life Cycle Costs for edge beams of three Norwegian bridges

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Abstract

For the planning of cost-effective infrastructure maintenance, it is important to assess the condition of structures and to estimate the optimum time, the extent and the type of repair needed to continue operating the structures safely. Today, reliable engineering tools for these tasks are lacking. Sometimes, structures with the highest degree of damage receive the highest priority in maintenance planning, and maintenance strategies tend to be reactive. In the context of more proactive maintenance, predictive modeling tools are needed in order to forecast the time when repair will be most cost-effective.

In this study, a novel approach is presented, which combines data from condition assessment of reinforced concrete structures (e.g. chloride profiles), predictive modeling (future chloride ingress) and experimentally determined chloride threshold values (C_{crit}) on cores retrieved from the structures with a method recently developed at ETH Zurich, Switzerland. This approach allows the prediction of the time of corrosion initiation. Furthermore, costs for different repair strategies are compared in order to provide a basis for maintenance planning.

This approach is applied to three bridges in Norway. The three investigated cases reveal that a condition assessement based on chloride measurements alone does not necessarily permit to assess the corrosion risk for a specific structure. In combination with measurements of C_{crit} for the actual structures, a more refined assessment of the corrosion risk can be done. This allows modeling the performance of the structure over the coming decades and thus provides the basis for selecting the most economic maintenance strategy.

Table of Content

1 Introduction

Infrastructure maintenance is one of the main challenges in civil engineering. Determining the best timing, the extent and type of repair depending on the condition of a structure is not straightforward. Thus, research is conducted to optimize existing maintenance plans. In this project, the service life is modelled for three existing bridges in Norway. The critical chloride content is determined on samples retrieved from the structures according to a method developed at ETH Zurich [1]. Model assumptions for chloride content and chloride diffusion coefficient were based on data available from condition assessment and laboratory testing. This allowed the calculation of the remaining service life and calculate all interventions costs to determine an optimal interventation strategy [2]. With this information, bridge maintenance can be optimized.

2 Available Documents

The following documents were available

- Research Agreement between ETH Zurich and Statens Vegvesen (dated 15.11.2018)
- Note Sampling for all three bridges from Statens Vegvesen
- Price List from Statens Vegvesen

3 Fundamentals

Chloride-induced corrosion is known as the main deterioration mechanism for reinforced concrete structures, especially in marine and road salt exposure. There is a strong need of infrastructure maintenance planning to avoid unnecessary costs and to guarantee the safety of a structure.
The chloride-ingress of uncracked concrete structures can be modelled by Fick's 2nd law of diffusion [3]. It is assumed, that corrosion initiates when a certain concentration of chlorides is reached in the concrete at the level of the reinforcement. This concentration of chlorides needed to trigger corrosion is generally termed the critical chloride content C_{crit}, or chloride threshold value. C_{crit} is known to exhibit extremely large variability in the literature [4-6]. Thus, to allow for making predictions for a specific structure, it is advisable to determine C_{crit} on samples from the actual structure.
By defining a level of corrosion probability as the theoretical end of service life (here 10%), the remaining service life can be modelled and intervention costs can be calculated much more precisely than relying on more general assumptions.

4 Case Studies

4.1 Haug Bridge

The Haug bridge, near Hokksund, is located approx. 60 km south-west from Oslo. The samples for the C_{crit} test were taken in 2019. Additionally, concrete cores for measuring the chloride profile were sampled (see Figure 1).
The bridge was built in 1988. The concrete mix contained probably 20% of fly ash. No information on the w/b ratio or silica fume was available.
The structural element was the northern and southern cantilever, as can be seen in Figure 1.

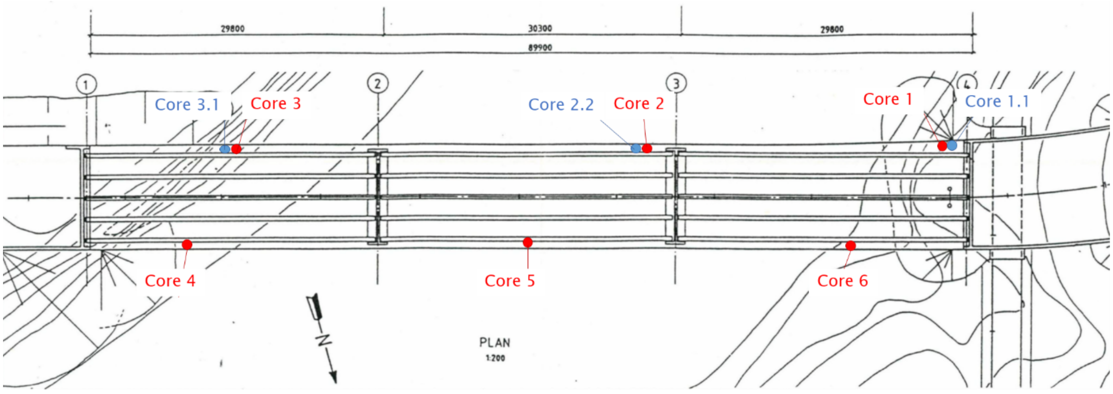

Figure 1 - Locations for sampling (drawing not according to scale). The red circles indicate the sampling for the C_{crit}-test, whereas the blue circles indicate the sampling for chloride profiling (from [7]).

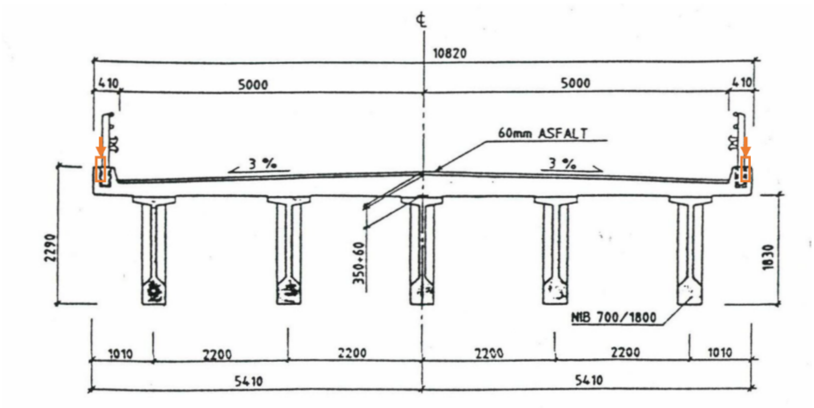

Figure 2 - Locations for sampling (drawing not according to scale) in the cross section (from [7]).

In addition to taking the concrete cores, the concrete resistivity and potential were measured and can be found in [7].

4.2 Straumhol Bridge

The Straumholbru bridge is located in Trøndelag (Osen kommune) about 140 kilometers north of Trondheim on the coast. The samples for the C_{crit} test were taken in 2019. The bridge was built in 1978. The concrete mix contained probably only ordinary Portland cement. No information on the w/b ratio or supplementary cementitious material was available.

The structural element was the eastern and western cantilever, as can be seen in Figure 3.

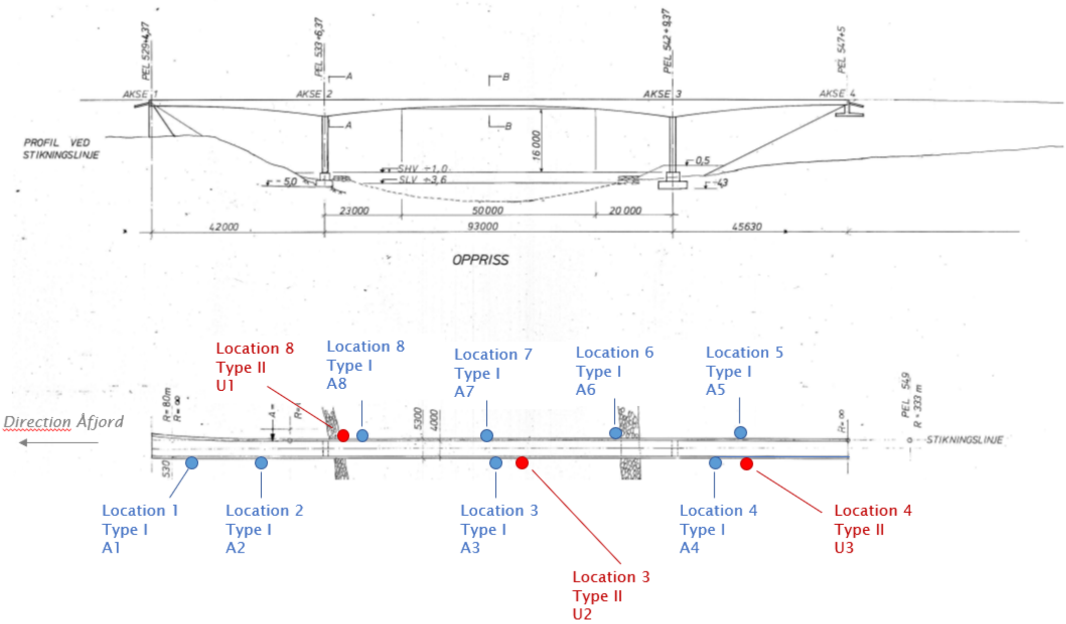

Figure 3 - Locations for sampling (drawing not according to scale). The blue circles indicate the sampling for the C_{crit}-test, whereas the red circles indicate the sampling for chloride profiling (from [8]).

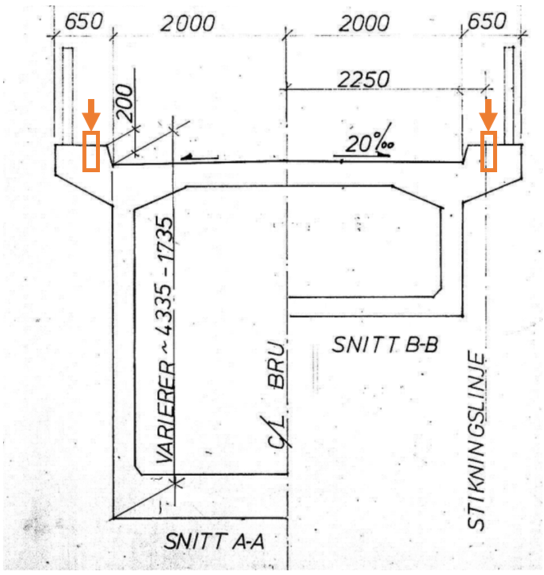

Figure 4 - Locations for sampling (drawing not according to scale) in the cross section (from [8]).

Concrete resisitivity was measured and can be found in [8].

4.3 Valsøy Bridge

The Valsøy bridge, in Trøndelag, is located about 120 km south-west from Trondheim. The samples for the C_{crit} test were taken in 2019. The bridge was built in 1992. The

concrete mix contained probably 20% of fly ash. No information on the w/b ratio or silica fume was available.
The structural element was the northeastern cantilever, as can be seen in Figure 1. The cantilever contained a hollow part inside the cantilever. The cover to the outside (chloride exposed surface) showed a higher cover than the cover to the inside (hollow part).

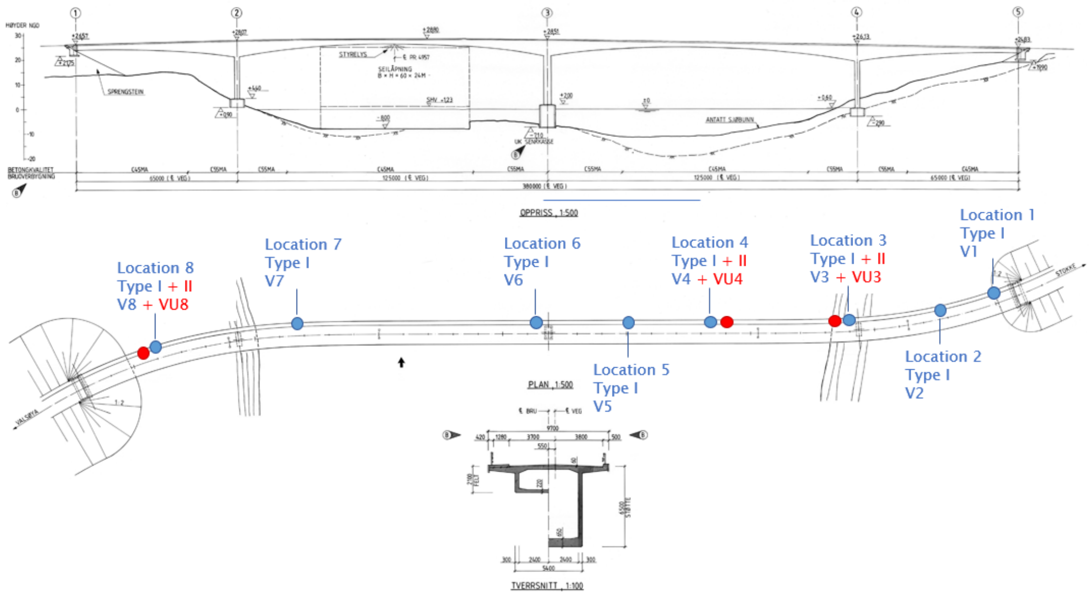

Figure 5 - Locations for sampling (drawing not according to scale). The blue circles indicate the sampling for the C_{crit}-test, whereas the red circles indicate the sampling for chloride profiling (from [9]).

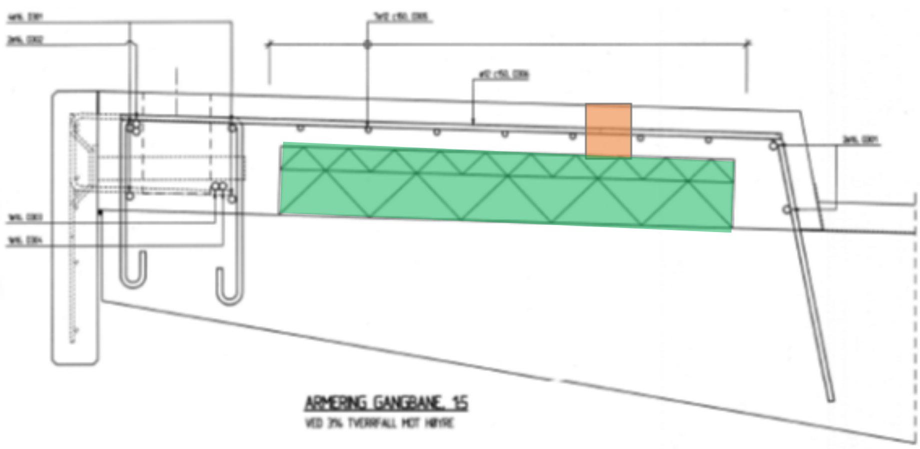

Figure 6 - Locations for sampling (drawing not according to scale) in the cross section. The hollow part is marked in green (from [9]).

5 Methods

5.1 Determination of C_{crit} in the laboratory

To determine C_{crit} in the laboratory, at least six samples per structural element should be taken. Each sample should contain one centrally located reinforcement, which is not yet corroding (to be verified with potential measurements).
Here, a short description of the experimental procedure is provided. A more detailed version can be found in [1].
The cover depth of the drilling cores take from each structure (diameter 120mm) was reduced by water-cooled diamond cutting to 15 mm to shorten the corrosion initiation time during later laboratory testing.
A durable electrical connection to the reinforcing steel was made by a selftapping screw and cable lug. The concrete at the steel bar ends was gently removed and refilled with a highly alkaline and dense mortar to prevent corrosion at the side faces. Subsequently, all lateral faces and the end parts of the exposed surfaces were coated with an epoxy based resin to guarantee uniaxial chloride ingress.
The samples were placed in a NaCl-solution (10% NaCl) and the steel potential was continuously logged versus an external reference electrode ($Ag/AgCl_{sat}$) to detect corrosion initiation. The criteria for corrosion initiation is a drop of potential of 150 mV and stability at this lower level for at least 7 d. The samples were then removed from the exposure solution, split, and the total chloride content was determined at level of reinforcement by dissolution in nitric acid, followed by titration, according to Swiss Codes [10].
The measured chloride contents for each bridge correspond to the statistical distribution of C_{crit} for this specific bridge.
The statistical parameters λ und ϵ^2 for a lognormal distribution were fitted and with the following formulas [11] converted in mean value μ_x and standard deviation σ_x of a normal distribution.

$$\mu_x = exp\left(\lambda + \frac{1}{2}\epsilon^2\right) \tag{1}$$

$$\sigma_x{}^2 = \mu_x{}^2 \times (exp(\epsilon^2) - 1) \tag{2}$$

5.2 Szenarios

For each structure, the corrosion probability was calculated with parameters directly measured for each specific structure.
For comparison, also a scenario using the statistical distribution of C_{crit} from the fib model code was taken (here simplified as a normal distribution with the following parameter: mean = 0.6, st.dev. = 0.15) [3].

5.3 Modelling of Service Life

5.3.1 Monte-Carlo Simulation

The service life of each bridge was modelled with a Monte-Carlo simulation using Fick's 2nd law of diffusion:

$$C(x,t) = C_s \times \left(1 - erf\left(\frac{x}{2 \times \sqrt{D_{app,0} \times \left(\frac{t_0}{t}\right)^n \times t}}\right)\right) \tag{3}$$

And the corresponding parameter: surface chloride concentration C_s, cover depth x, apparent diffusion coefficient $D_{app,0}$ at time t_0, and age exponent n.
The surface chloride concentration was extrapolated to the surface by using the measured chloride profiles.
Each parameter is described in chapter 5.3.2 Parameter.
The result is the chloride concentration $C(x,t)$ at level of reinforcement x at time t and is calculated for each year.
The probability of corrosion is calculated by using a Monte-Carlo simulation, viz. for each time step (generally 1 year), 1'000'000 random numbers were sampled from the respective statistical distributions of each model parameter. This led to 1'000'000 calculations of chloride contents $C(x,t)$ at level of reinforcement per year. These were compared to 1'000'000 randomly chosen C_{crit} of the corresponding statistical distribution of C_{crit}. Values below zero indicated corrosion, values above zero indicated no corrosion. The probability was the ratio between corrosion and no-corrosion events.
With the inverted standard normal distribution the parameter beta is calculated, which is commonly used in life cycle modelling:

$$\beta = -\Phi^{-1}(P_{corr}) \tag{4}$$

The reliability index β is often assumed in the range 2.0 to 0.5, which corresponds to 2% to 30% of probability for corrosion initiation [12]. The fib model code for service life design suggests minimum reliability indices of 1.3 (approx. 10% probability) for depassivation (serviceability limit state) and 3.7–4.4 for ultimate limit state (e.g. collapse).
In this study, a corrosion initiation probability of 10% is chosen (reliability index = 1.3).

5.3.2 Parameter

5.3.2.1 C_{crit}

A lognormal distribution was fitted to the measured C_{crit}. The results of the fitting as well as the measured C_{crit} are given in the Table 1 and depicted in Figure 7. For the service life modelling the fitted values are used as input parameters.
It can be seen that the distributions of C_{crit} vary considerably between the three different structures. The bridge Haug has comparatively low C_{crit}, with an average value around 0.65%Cl per cem. wt. The other two structures seem to have considerably higher C_{crit}, but also much higher scatter.
The cement type could not explain the low values for the Haug bridge (most likely 20% of fly ash added).
The variability of C_{crit} for the three different structures may seem high when compared to well-known literature data or the data given in the fib model code for service life design [3]. This high variability in comparison to established C_{crit} can be explained by the actual conditions in samples taken from structures, as compared to the more well-controlled laboratory samples (mortar, homogeneous material properties, etc.) that led to C_{crit} distributions as the one in the fib model code for service life design [3].

Table 1 - Statistical parameters for lognormal distributions of C_{crit} for all three bridges. All numbers in %Cl per cem. wt.

Bridge	$\lambda = \mu_{\ln x}$	$\epsilon = \sigma_{\ln x}$	μ_x fit	σ_x fit	$\mu_x$meas	σ_x meas	values
Haug	-0.9	1.00	0.67	0.88	0.65	0.84	0.08; 0.08; 0.19; 0.44; 0.6; 2.48
Straum-hol	-0.3	1.2	1.52	2.73	1.87	2.31	0.3; 0.32; 1.02; 5.83
Valsøy	0.85	0.48	2.63	1.34	2.68	1.27	1.34; 1.4; 1.46; 2.25; 3.91; 3.98; 4.4

For Haug and Valsøy, the statistical distribution fit well to the measured values. For Straumhol, the fitting is more difficult, as one of the measured C_{crit} is much higher than the rest of the values. The here fitted statistical distribution takes this high C_{crit} into account.

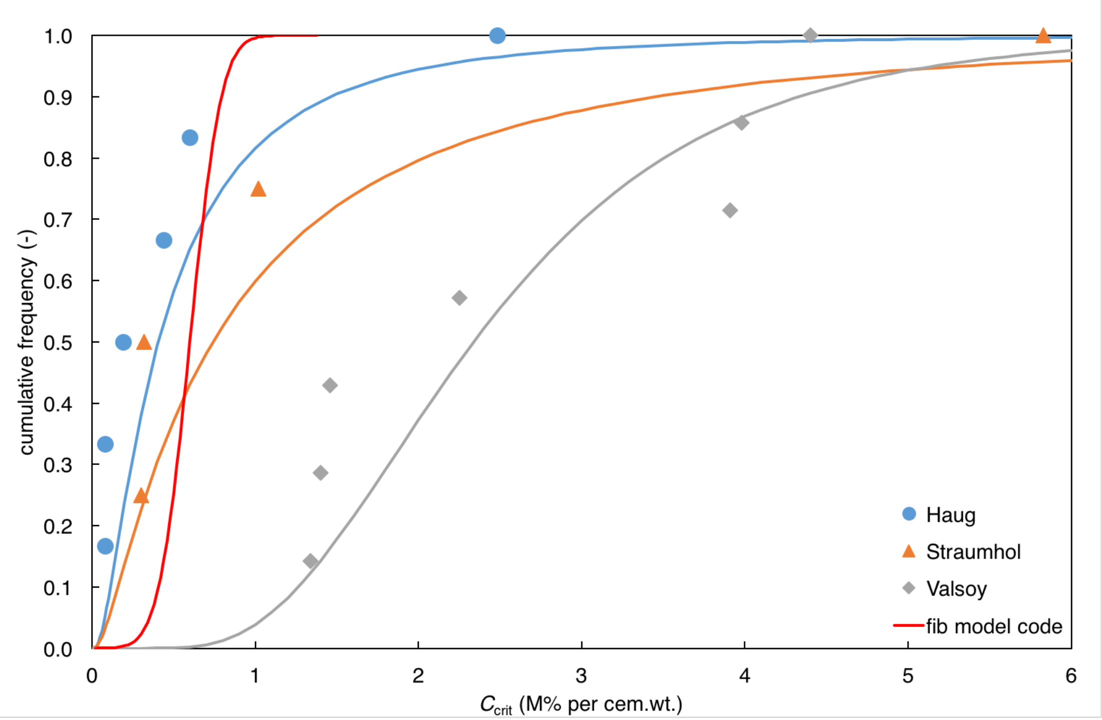

Figure 7 - C_{crit} distribution (measured and fitted) for all three structures. In red, the cumulative C_{crit} distribution from the fib model code [3] is depicted.

5.3.2.2 Surface chloride concentration, diffusion coefficient, and concrete cover

These three parameters were determined for each bridge:

- Chloride surface concentration: The chloride profile was measured for each drilling core without reinforcement, the results of the chloride profiles can be found in Appendix A. By fitting the chloride concentrations with formula 1 to the measured chloride profiles (x=0), C_s can be determined.
- Diffusion coefficient: Two methods were used to determine the diffusion coefficient:
    - Method 1: The apparent diffusion coefficient for the actual age of the structures was estimated from the concrete resistivity, which was measured on the samples for C_{crit} test after splitting. A mean value of the conversion factors from the following references were used: [13-17]. Note that here, the resistivity measured at the end of the laboratory test related to a relatively moist condition of the concrete and with a relatively high chloride content. Thus, method 1 may lead to an overestimation of the chloride diffusion coefficient as it was in the structure. Due to this overestimation of the diffusion coefficient, the modelled service lives would be unrealistically short. The results of the service life modelling is added, but not used for the calculation of the costs.
    - Method 2: By fitting the chloride concentrations with formula 1 to the measured chloride profiles (x=0), D_0 can be determined. This approach may reflect better than method 1 the conditions relevant for chloride transport in the structure. In this case, n was assumed to be 0.4 and t

was assumed according to the age of the bridge at the time of chloride sampling.

- Cover depth: the cover depth was measured on each samples for the C_{crit} test. The mean was calculated for each bridge.

The results of these appraoches to determine the parameters chloride surface concentration, diffusion coefficient, and cover depth are summarized for each bridge in Table 2 to Table 4.

Table 2 - Statistical parameters for chloride surface concentration (at x=0 mm), diffusion coefficient, and cover depth for the Haug bridge (age = 31).

Parameter	mean value	st.dev.	values
C_s (% per cem.wt.)	5.5	1.5	7.0; 6.0; 3.5
D_0 (10^{-12} m^2/s) (method 1)	16.8	2.7	805; 699; 641; 774; 1059*
D_0 (10^{-12} m^2/s) (method 2)	2.2	0.57	1.9; 3.0; 1.7
x (mm)	61	6	53; 55; 63; 71; 62; 63

* concrete resistivity values (in Ohm*m) measured after splitting

Table 3 - Statistical parameters for chloride surface concentration (at x=0 mm), diffusion coefficient, and cover depth for the Straumhol bridge (age = 41).

Parameter	mean value	st.dev.	values
C_s (% per cem.wt.)	1.3	0.8	1.4; 0.3; 2.3
D_0 (10^{-12} m^2/s) (method 1)	127	41.8	75; 97; 181*
D_0 (10^{-12} m^2/s) (method 2)	4.1	0.69	3.8; 3.5; 5.1
x (mm)	50	14	55; 51; 64; 30

* concrete resistivity values (in Ohm*m) measured after splitting

Table 4 - Statistical parameters for chloride surface concentration (at x=0 mm), diffusion coefficient, and cover depth for the Valsøy bridge (age = 27).

Parameter	mean value	st.dev.	values
C_s (% per cem.wt.)	1.3	0.2	1.3; 1.6; 1
D_0 (10^{-12} m^2/s) (method 1)	95.8	25.8	206; 123; 167; 125; 171; 136; 88*
D_0 (10^{-12} m^2/s) (method 2)	3.9	1.0	2.4; 4.8; 4.4
x (mm)	68	13	72; 74; 75; 80; 80; 80; 83; 58; 62; 50; 54; 45

* concrete resistivity values (in Ohm*m) measured after splitting

5.4 Service Life of Intervention Strategies

5.4.1 Hydrophobic Treatment

It is assumed, that a hydrophobic treatment prevents the ingress of chloride-contaminated water completely for 10 years. After that time interval, the protection against water ingress is assumed to be zero. This is a simplification, as in reality the ingress of water and chlorides might still be reduced to a certain amount.

5.4.2 Classical concrete repair

It is assumed, that the classical concrete repair, by removing the old cover and replacing it with a repair mortar or concrete, itself has a limited life time. This value is assumed to be 15 years and is based on experiences [18]. However, this assumption leads to very high repair costs in cases were the concrete repair has to be repeated several times in an interval of 15 years to achieve the 100 y design service life. Thus, in the presence of experience deviating from the one reported in [18], that is, longer lifes of the concrete repair, e.g. due to lessons learned from the past and improved repair quality (particularly the bond between old and new concrete), one may justify assuming a longer life of the repair. Here, we thus also calculate a case with 30 years of life of the concrete repair for comparison.

5.4.3 Cathodic Protection

The service life of cathodic protection is assumed to be until the end of the service life of the structure itself.

5.5 Cost Estimation

Life cycle costs consist of the new reconstruction of the structural element as well as probable repair work during the defined service life (viz. 100 years). Demolition is not included in this calculation. Also not included are ecological or social costs.
The probable repair work, which might be needed in the future, has to be corrected by the interest rate for capitlization (here 1.03) and the interest rate inflation (here 1.02).
As in this work, the structures are already built, the costs for construction are not included in the calculation.
The following formula (from [2]) is here used for the calculation of the service life costs.

$$K = K_E + \sum_{i=1}^{n}\left(\left(K_{t,i} + K_v\right)\times {q_1}^{-ti}\times {q_2}^{ti}\right) \qquad (5)$$

with the following parameters: costs for construction K_E (=0, as the structures are existing and the construction is not included in the comparison of intervention strategies), costs at time i due to repair work $K_{t,i}$ (summarized in Table 5), costs due to installation of traffic interruption or necessary detours K_v (summarized in Table 5), interest rate capitalization q_1=1.03, and interest rate inflation q_2=1.02. All costs included in this calculation have to be paid by the owner.

5.5.1 Costs for construction

The cost estimation is based on the price list provided by Statens Vegvesen, which are based on experience.

Table 5 - Average prices for Norwegian bridge maintenance

Construction Costs	**Price**	**Prices from contractor**
Concrete (incl. Casting)	(NOK / dm^3)	10
Mortar with hand application	(NOK / dm^3)	40 – 80
Dry sprayed mortar	(NOK / dm^3)	25 – 50
Surface treatment (incl. Surface preparation)	(NOK / m^2)	200 – 300
Reinforcement (replacement)	(NOK / m)	200 – 300
New reinforcement	(NOK / t)	50'000 – 60'000
Hydrodemolition (below level of reinforcement)	(NOK / m^2)	1’500-2’000
Sandblasting	(NOK / m^2)	100 – 250
Demolition of reinforced concrete (mechanic)	(NOK / dm^3)	60 – 80
Cathodic protection (installation costs)	(NOK / m^2)	2'000 – 4’000
Cathodic protection (operational costs)	(NOK / year / system)	approx. 20’000
Simple mechanical repair (with water chiseling (hydrodemolotion) and dry spraying)	(NOK / dm^3)	250 – 500 (more expensive the larger the bridge)
Replacement of cantilever and railing (removing concrete until the reinforcement in the deck, reconstruction)	(NOK / m)	12'000 – 18’000
General Costs	**Price**	**Prices from contractor**
costs for infrastructure because of detour traffic signs etc.	(% of total costs)	approx.. 25%

6 Results

6.1 Service Life

The calculated service lives for each bridge are given in Table 6 and Figure 8, Figure 9, and Figure 10. The service life includes the time to corrosion initiation with the defined corrosion probability. It does not consider any measures against chloride ingress or corrosion inhibitors. The tables below always show the time to corrosion initiation for C_{crit} as determined for the specific structure ("C_{crit} structure") and for C_{crit} taken from the fib model code for service life design ("C_{crit} code").

Table 6 - Calculated service life in years with C_{crit} from structure and C_{crit} from the fib model code [3]. With D_0 estimated on the basis of the concrete resistivity measured at the end of the laboratory tests.

Bridge	**C_{crit} structure**	**C_{crit} code**
Haug	3	5
Straumhol	1	1
Valsøy	>> 1'000	3

As already described in Chapter 5.3.2.2, the diffusion coefficient determined with method 1 overestimates the "real" diffusion coefficient, most likely due to the high chloride concentrations and high concrete moisture and accordingly low resistivity. The results of the service life modelling are therefore unrealistically short – and are not further considered in this report.

Table 7 - Calculated service life in years with C_{crit} from structure and C_{crit} from the fib model code [3]. With D_0 from the chloride profiles.

Bridge	**C_{crit} structure**	**C_{crit} code**
Haug	66	125
Straumhol	39	72
Valsøy	>> 1'000	435

It can be observed, that the service life with a C_{crit} according to the fib model code [3] leads to higher service lifes than the C_{crit} from structures Haug and Straumhol. The Valsøy Bridges shows higher C_{crit} than the fib model code [3] and therefore also longer service lifes.
From the calculated service lives, the age of the structure has to be substracted, which will lead to the following remaining service lives.

Table 8 - Calculated remaining service life in years with C_{crit} from structure. With D_0 from the chloride profiles.

Bridge	**remaining service life**
Haug	35
Straumhol	-2
Valsøy	>> 1'000

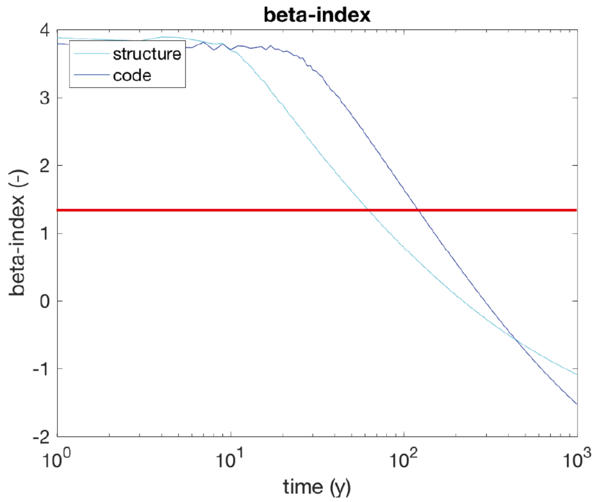

Figure 8 - Reliability index over time for the Haug bridge.

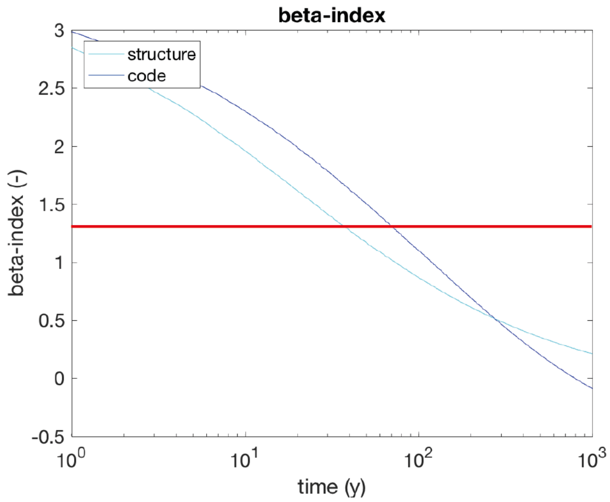

Figure 9 - Reliability index over time for the Straumhol-Bridge.

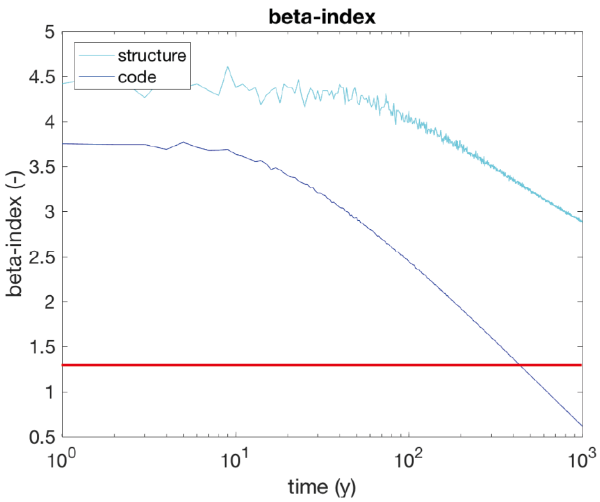

Figure 10 - Reliability index over time for the Valsøy-Bridge.

The results in Table 7 and Figure 10 show that corrosion will not initiate during the service life at the Valsøy Bridge. This is due to the combination of high cover depth, relatively low surface chloride concentration and comparatively high C_{crit}.

Taking into account the ages of the bridges, the remaining predicted service life for the Haug Bridge is approximately 35 years (Figure 11), whereas for the Straumhol Bridge the remaining service life is less than 0 years (-2 years). These values are given in Table 8. This means, that the 10% corrosion probability was already exceeded 2 years before the sampling (thus, in 2017). Note that the corrosion probability now (2022) is, however, only slightly higher than 10% (Figure 12). As corrosion has already initiated at the Straumhol Bridge, hydrophobic treatment cannot any longer be used as intervention strategy to restore passivity and fully arrest corrosion. The further reduction of chloride and moisture ingress may not sufficiently slow down the corrosion rate.

In the following figures, the increase in the corrosion probability over time can be observed and the remaining service life is indicated.

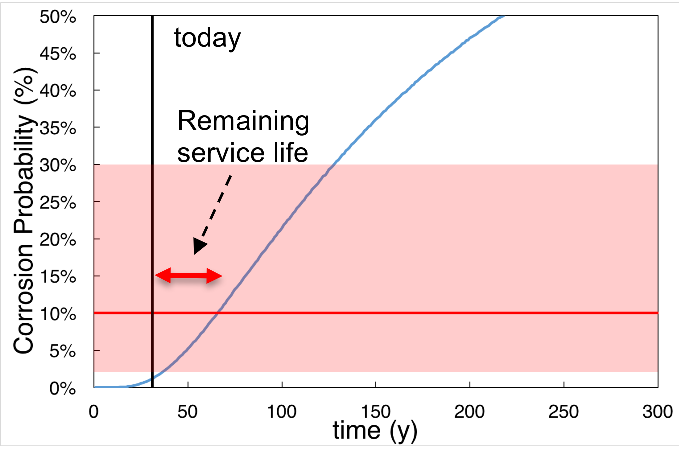

Figure 11 - Corrosion probability over time for the Haug Bridge. The light shaded area indicates the range of commonly accepted probability of corrosion.

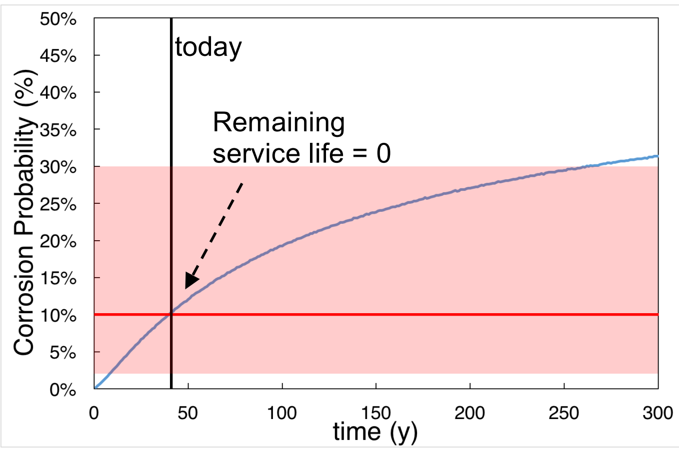

Figure 12 - Corrosion probability over time for the Straumhol Bridge. The light shaded area indicates the range of commonly accepted probability of corrosion.

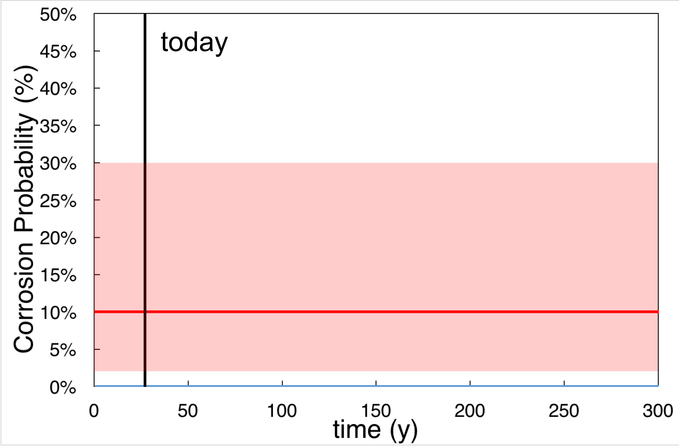

Figure 13 - Corrosion probability over time for the Valsøy Bridge, it is until 300 years negligibly low and is indicated by the blue line at the x-axis. The light shaded area indicates the range of commonly accepted probability of corrosion.

6.2 Costs

This section presents the costs according to the calculations described in detail in section 5.5.
As the Valsøy Bridge does not need any intervention to reach its service life of 100 years (it takes way longer for corrosion to initiate), there are no additional costs (additional to the constructions costs) to extend the service life of the bridge.
For the Haug and Straumhol Bridge at least one intervention is needed, summarizes the amount of needed interventations per strategy and bridge. Table 10 summarizes the cost for the different possible intervention strategies. Costs due to installation and traffic detours are included with 25%.

Hydrophobic treatment is not possible anymore for the Straumhol Bridge, as corrosion has probably already initiated.

Table 9 - Remaining service life per bridge and amount of needed interventions.

Bridge	concrete repair	hydrophobic treatment	cathodic protection
Haug (remaining service life 35 y)	3* (2)**	4	installation and 33 y operational costs
Straumhol (remaining service life 0 y)	4* (2)**	-	installation and 58 y operational costs

* The concrete repair is planned to be renewed every 15 years.
** The concrete repair is planned to be renewed every 30 years.

Table 10 - Accumulated repair costs for the remaining service life for Haug and Straumhol bridge (NOK per m edge beam).

Bridge	concrete repair	hydrophobic treatment	cathodic protection
Haug	7’930* (5'306)**	1’056	3’038
Straumhol	13’914* (7’465)**	-	4’275

* The concrete repair is planned to be renewed every 15 years.
** The concrete repair is planned to be renewed every 30 years.

Hydrophobic treatment is the cheapest intervention strategy for the Haug Bridge. To guarantee effectiveness of the hydrophobic treatment, it should be tested and renewed regularly (Here, 10 years were assumed).
For the Straumhol Bridge, cathodic protection is the cheapest intervention strategy, as corrosion has already initiated.

Concrete repair is in all cases the most expensive intervention strategy, even if it is only renewed every 30 years.

7 Discussion

7.1 Applicability of test method

The here proposed test method to determine C_{crit} at structures is easily applicable to structural elements and leads to more reliable results for determination of service life design than the assumption of a generic, fixed C_{crit} such as 0.4% chloride by cement weigth. Nevertheless, the method is, depending on exposure conditions in the lab, concrete resistivity (chloride ingress) and chloride concentration at level of reinforcement, sometimes time consuming. For this reason, tests for the condition assessment should be planned well in advance to avoid delays in the decision taking for repair strategies. This should be considered in the long-term planning of maintenance strategies for infrastructure owners.

The costs of the test method for C_{crit} are relatively low compared to the possibility of reducing costs due to unecessary repair. Moreover, the reduction of underestimated corrosion risk leads to earlier intervention (when repair costs are still low), and higher safety for infrastructure users (though failure due to corrosion might also be noticed in other condition assessment strategies).

7.2 Structure dependent C_{crit}

The results show, that the structure dependent C_{crit} is not comparable in between structures or to one fixed C_{crit} given in codes or text books. It is therefore worthwile to measure C_{crit} for each structure to determine the corrosion probability for each structure. The three cases investigated in this study reveal that a condition assessement based on chloride measurements alone would not necessarily have permitted to reliably assess the corrosion risk for a specific structure. In combination with measurements of C_{crit} for the actual structures, a more refined assessment of the corrosion risk could be done.

8 Conclusions

It is worth to point out the following general conclusions:

- To reduce the costs for concrete repair and maintenance work, it is useful to start the maintenance work, before corrosion started to initiate. Then, the cheapest maintenance strategy (hydrophobic treatment) can be chosen.
- High quality concrete and high cover helps to reduce the risk of corrosion over the service life of a structure.
- The three investigated cases reveal that a condition assessement based on chloride measurements alone does not necessarily permit to assess the corrosion risk for a specific structure. In combination with measurements of C_{crit} for the actual structures, a more refined assessment of the corrosion risk can be done. This allows modeling the performance of the structure over the coming decades and thus provides the basis for selecting the most economic maintenance strategy.

For the specific cases, which has been chosen for this report, the following conclusions can be drawn:

- For the Valsøy Bridge, no corrosion is expected to initiate during the service life. This would not be apparent in a condition assessement only based on chloride profiles. As the C_{crit} is higher than the value given in the fib model code for service life design and similar documents, the service life is determined to be so long.
- The Straumhol Bridge needs immediate action (within the next 5 years) as the defined threshold probability for corrosion initiation has already been exceeded. The installation of a cathodic protection system could limit further corrosion damage and guarantee the serviceability until the end of service life. The CP is more cost efficient than the concrete repair.
- The Haug Bridge has not yet reached the defined threshold probability for corrosion initiation, but is expected to do so before the end of service life. Corrosion will probably initiate within the coming decades. Therefore, actions on the long term should be planned. Based on the cost calculations, the application of a hydrophobic treatment appears to be the most economic strategy, as it could reduce the amount of chlorides entering through the cover.

These detailed intervention strategies can be now tailored for each structure as the C_{crit} was determined for each structure and a service life cost analysis had been performed.

31. March 2022,

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Appendix A – Chloride Profiles

Haug Bridge

sample ID	cover depth (mm)	chloride content (% per cem.wt.)
1-1	0-10	1.48
	10-20	3.32
	20-30	1.13
	30-40	0.27
	40-50	0.07
	50-60	0.04
2-2	0-10	1.08
	10-20	2.02
	20-30	1.87
	30-40	0.97
	40-50	0.14
	50-60	0.04
3-1	0-10	1.57
	10-20	1.46
	20-30	0.45
	30-40	0.11
	40-50	0.05
	50-60	0.04

Straumhol Bridge

sample ID	cover depth (mm)	chloride content (% per cem.wt.)
U1	0-10	1.12
	10-20	0.88
	20-30	0.64
	30-40	0.32
U2	0-10	0.23
	10-20	0.19
	20-30	0.12
	30-40	0.06
U3	0-10	0.44
	10-20	0.90
	20-30	0.96
	30-40	0.75
	40-50	0.21

Valsøy Bridge

sample ID	cover depth (mm)	chloride content (% per cem.wt.)
VU3	0-10	0.68
	10-20	0.59
	20-30	0.32
	30-40	0.16
VU4	0-10	0.78
	10-20	0.90
	20-30	0.69
	30-40	0.37
VU8	0-10	0.81
	10-20	0.59
	20-30	0.41
	30-40	0.22

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