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LIFE CYCLE DESIGN OF CONCRETE STRUCTURES

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Abstract

For all concrete structures where safety and a reliable long-term performance are important, a reliability-based life cycle design should be carried out. As part of the life cycle design, calculations or assessments of life cycle costs and life cycle ecology should also be included in the basis for decision-making. In the present paper, the fundamentals for a reliability-based life cycle design are briefly outlined. Assessment procedures for life cycle costs are also briefly included and applied as an example to the repair of a heavily corroding concrete harbor structure. This example demonstrates that a proper utilization of stainless steel reinforcement would originally have been a very good investment and strategy for obtaining a good long-term performance compared to that of the traditional design applied with the corresponding repair and maintenance costs.

1. Introduction

Although deteriorating processes such as expansive alkali-aggregate reactions (AAR) and freezing and thawing also represent severe durability problems, it is not the disintegration of the concrete itself but rather electrochemical corrosion of embedded steel which poses the most critical and greatest threat to the durability and long-term performance of concrete structures [1]. In particular this is true for concrete structures exposed to chlorides. Therefore, most of the research and development on durability design that has been carried out in recent years has been related to concrete structures in chloride containing environments.

Basically, the time to depassivation is most commonly modelled by Fick's 2nd Law in combination with a time dependent diffusion coefficient [2]. However, since the parameters both for concrete durability and environmental exposure typically show a high scatter, the introduction of a reliability-based life cycle design has proved to be very valuable [3-6]. Although there is still a lack of relevant data, this methodology has already been successfully applied to several new concrete structures, where requirements to a more controlled durability and service life have been specified.

In order to include the uncertainty of the various parameters involved, a similar approach as that used for structural design is normally applied. In principle, it is a question of evaluating structural reliability and probability of failure. As part of the life cycle design, calculations or assessments of life-cycle costs does also provide an improved basis for decision-making.

2. Structural reliability

The long-term performance of a structure can be expressed in the form of a reliability format. The reliability of a structure or a component of the structure is defined as its probability of survival (p_s), which is related to the probability of failure (p_f) by:

$$reliability = p_s = 1 - p_f \tag{1}$$

Failure is defined in relation to different possible failure modes referred to as limit states. The ultimate limit state represents the inability of a structure to resist the imposed load effects. The serviceability limit states are also defined as the inability of the structure to meet its normal use or durability requirements. When the structure has deteriorated to a certain level where it will not longer function to the required reliability level, the end of service life or ultimate limit state (ULS) is reached. By carrying out repairs, it is possible to increase the structural performance and reduce the rate of degradation in such a way that the time for ULS will be extended. If the repair is un-successfully carried out, however, the rate of degradation may accelerate. This may be the case for badly performed patch repairs of chloride-induced corrosion damage [7].

In order to meet the operational requirements, the probability for exceeding *all* limit states during service life must be kept within a set of pre-determined performance limits. These limits define the performance requirements for a structural component -- or the entire structural system. The performance of any particular component, sub-system or system of an asset depends upon a number of variables. If the stochastic processes defining the residual strength and the probabilistic characteristics of the loads at any time are known, the failure probability of a structural component can be evaluated as a function of time. Both the strength R(t) of the structure and the applied loads S(t) can be expressed as a distribution function of time, where the structural resistance is a cumulative distribution function (CDF) and the load is a probability distribution function (PDF). At any time (t) the margin of safety, M(t), is:

$$M(t) = R(t) - S(t) \tag{2}$$

If R and S are statistically independent, the probability of failure is:

$$p_f = p[M(t) < 0] = \int_0^\infty F_R(x) \cdot f_S(x) dx \tag{3}$$

where: $F_R(x)$ .... CDF of the resistance, R, and

$f_S(x)$ .... PDF of the load

The failure region is the region, where the distribution functions of the resistance and the load intersect (Fig. 1). The graphical presentation of an example of calculating the failure probability is shown in Fig. 2.

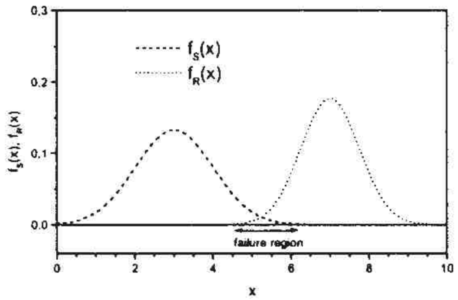

Fig. 1. Distribution functions of the resistance ® and the load (S). The failure region is indicated by the intersecting curves.

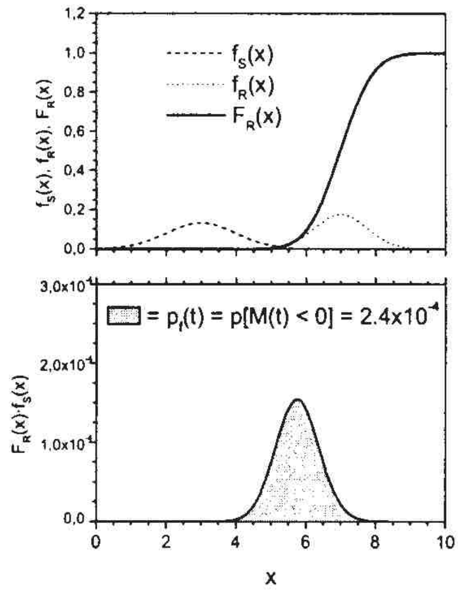

Fig. 2. Results of a calculation for failure probability (arbitrary values used).

As already indicated by the distribution functions, it is not easy to make a good long-term prediction of the structural performance, since the effects of various maintenance and rehabilitation options very much affect the results. In a decision making process, however, the impact of any repair and maintenance option upon the future performance of the structure has to be considered. While maintaining the structural reliability to an acceptable level, all costs involved should also be minimized. Therefore, calculations or assessments of life cycle costs are also an important part of the life cycle design.

3. Life cycle costs

3.1 General

Calculations or estimations of costs against benefits can be carried out in different ways by considering various costs or benefits, often referred to as a whole life cycle costing, cost-benefit or cost-benefit-risk analysis. Life cycle costs (LCC) may be used to assess the "cost-effectiveness" of

- Design decisions such as optimal durability requirements (cover, protective coatings, etc.)

- Construction quality
- Inspection, maintenance and repair strategies

The life cycle costs of a structure up to the time (t_N) may be represented as:

$$LCC(t_N) = C_I + C_{QA} + \sum_{i=1}^{t_N} \frac{C_{IN}(t_i) + C_M(t_i) + C_R(t_i) + \sum_{LS=1}^{M} p_{f_{LS}}(t_i) \cdot C_{f_{LS}}}{(1+r)^{t_i}} \tag{4}$$

where:
C_I .... design and construction cost
C_{QA}.......... cost of quality assurance and quality control
$C_{IN}(t)$....... expected cost of inspections
$C_M(t)$ .... expected maintenance costs
$C_R(t)$........ expected repair costs
M .... number of limit states, LS
$p_{f_{LS}}(t)$ ...... annual probability of failure for each limit state
$C_{f_{LS}}$ .... failure costs associated with the occurrence of each limit state
r............... discount rate

However, this representation of life cycle costs fails to account for the advantage of designing or maintaining structures to have a longer service life. Alternatively, it may be more meaningful, therefore, to compare costs on an annual-equivalent basis by distributing life cycle costs over the whole lifetime of the structure by an annuity factor, which expresses the annuity or annual costs. The average annuity cost (C_A) during the service life of a structure (n years) is:

$$C_A(t_N) = \sum_{j=1}^{n} \frac{p_f(t_j) \cdot r \cdot [C_I + C_{QA} + C_{IN}(t_j) + C_M(t_j) + C_R(t_j)]}{1-(1+r)^{-t_j}} \tag{5}$$

where $p_f(t_j)$ represents the probability of failure in year (j) and

$$p_f(t_n) = 1 - \sum_{j=1}^{n-1} p_f(t_j)$$

Experience has shown that calculations of annuity costs represent a good way of expressing increased investment costs for increased durability. Calculations of life cycle costs may also include other costs and benefits such as traffic delays or reduced travel time, efficiency of inspections, maintenance and repair strategies etc. Evidently, the decision analysis should be the subject for a sensitivity analysis in order to ensure that decisions are not unduly influenced by uncertainties in structural reliabilities and damage, construction or other costs.

3.2 Life cycle costs of a corroding concrete harbor structure

In order to demonstrate how an assessment of life cycle costs may provide an improved basis for decision making of various technical solutions for improved durability, a heavily corroding concrete structure in a Norwegian harbor is used as an example. The harbor structure, which was an open structure with a concrete deck of 132 m × 17 m on top of tremie-cast concrete pillars, was constructed in 1964. The deck consisted of 3 longitudinal main beams and 18 transversal secondary beams with two-ways slabs in-between. The main beams and the secondary beams had dimensions 90 cm × 120 cm and 70 cm × 70 cm, respectively, while the top slab was 25 cm thick including a 6 cm top layer. The mechanical loads on the deck consisted mainly of two heavy loading cranes, 60 tons and 100 tons, respectively, moving on top of the three longitudinal main beams.

After a service period of 38 years (2002), the general condition of the structure was very poor. A structural assessment confirmed that the load-bearing capacity of the main beams would only be acceptable for continued operation of the cranes for a very short period of time, and the rate of corrosion in these beams was so high that an immediate repair was needed. The concrete pillars were in fairly good condition, but the deck-slabs had reached such a high degree of deterioration that all other traffic on the deck had already been prohibited. Of the various technical solutions for repair considered, one option was simply be to construct a new concrete deck on top of the old deck. Since both the crane facilities and the structure would not be needed for a continued service and operation for more than further 15 years, the construction of a new deck would represent a very expensive solution. Therefore, a cathodic protection system was specially designed in order to extend the service life of the deck beams for a limited period of time [8, 9].

If the above structure originally had been the subject for analysis of life cycle design including life cycle costs, a more controlled service life would probably have been obtained. If the owners objective was to keep a safe operation of the structure for a service period of approximately 50 years, the following life cycle costs of various technical solutions could have been considered. Based on such a situation, some calculations of life-cycle costs, for various technical solutions are shown in the following. As a basis for the calculations, some basic information about the old structure was needed. Although it was not easy to collect all relevant information, some information was obtained as shown in Table 1.

Table 1. Basic information about the old structure.

Concrete quality:	45 MPa
Concrete cover in beams:	75 mm
Concrete cover in slabs:	25 mm
Assumed new construction costs:	25.000.000 NOK
Amount of concrete:	1532 m^3
New costs of concrete(1.200 NOK/m^3)	1.840.000 NOK
Amount of steel:	315 t
New costs of steel (3.650 NOK/t)	1.150.000 NOK
Material costs related to total costs:	
Concrete:	7.4 %
Steel:	4.6 %

In order to show the principles for cost calculations, the following alternative options were considered:

- Doing nothing than what was originally designed
- Using epoxy coated reinforcement
- Using stainless steel as reinforcement in beams (100 %)
- Partly using stainless steel as reinforcement in beams (75 %)
- Using stainless steel clad reinforcement in beams (100 %)
- Partly using stainless steel clad reinforcement in beams (75 %)
- Increasing concrete cover from 75 mm to 100 mm in beams
- Increasing concrete quality from 45 to 70 MPa
- Increasing concrete quality from 45 to 70 MPa in combination with increasing concrete cover from 75 mm to 100 mm in beams

In the following, the life cycle costs of all these options are compared for a service period of 50 years. For convenience, the discount rate was put to zero in all calculations. Annuity costs were therefore calculated by the total costs divided by the expected service life. Other maintenance costs that usually come along with a structure in service are not included.

a) Doing nothing

The total life cycle costs for this option was NOK 25.000.000. It was assumed that the service life would end after an extended period of 3 years (2005), which means that the annuity costs were calculated to approximately NOK 630.000.

b) Epoxy coated reinforcement

With a cost ratio for epoxy coated rebar to black steel of 1.5 [10]. the material costs for the reinforcement would be increased to NOK 1.730.000. This means increased total costs by 2.3 % to NOK 25.580.000. Also here it was assumed an extended service life of 3 years (2005). Recent experience on epoxy coated reinforcement has shown very mixed results, and some authorities do not any longer recommend the use of epoxy coated reinforcement for chloride containing environment [11]. Since epoxy coated rebars would not increase the service life very much, the annuity costs were calculated to less than NOK 640.000.

c) Stainless steel reinforcement

Stainless steel reinforcement has been known to perform very well in marine environment for a long time. Thus, in the Progreso Pier in Yucatan, Mexico, type AISI 304 stainless steel has performed very well during a period of more than 60 years in a hot, humid, and salty environment with virtually no maintenance [12]. In the literature, it is generally assumed that a proper use of stainless steel will increase the service life by a factor of at least two [13]. By using a cost ratio for stainless steel to black steel of 4.5 [13], the material costs for the reinforcement would increase to NOK 5.200.000, which again would increase the total costs by 16.1 % to NOK 30.200.000. With an assumed extended service life of approximately 40 years, the annuity costs were calculated to less than NOK 380.000.

d) 75 % Stainless steel reinforcement.

If the the stainless steel was only used in the most exposed parts of the structure, the amount of stainless steel could be significantly reduced. If the amount of stainless steel was only reduced by 25 %, the material costs for the reinforcement would be NOK 4.170.000, which would increase the total costs by 12.1 % to NOK 28.020.000. For an assumed extended service life of approximately 40 years, the annuity costs were calculated to less than NOK 350.000.

d) Stainless steel clad reinforcement

As an alternative to pure stainless steel, stainless steel clad reinforcement may also have been considered. This type of reinforcement consists of a carbon steel core with an 1–2 mm outer layer of stainless steel. Little information about the service life of such steel is available, but by assuming an extended service life of at least 20 years and a cost ratio to black steel by 3 [10], the material costs for the reinforcement would increase to NOK 3.450.000. This would increase the total costs by 9.2 % to NOK 27.300.000. For an assumed extended service life of 20 years, the annuity costs were calculated to approximately NOK 460.000.

e) 75 % Stainless steel clad reinforcement

By a similar reduction in the stainless steel clad reinforcement as for the pure stainless steel, the material costs for the reinforcement would be NOK 2.870.000, which would increase the total costs by 6.9 % to NOK 26.720.000. For an assumed extended service life of 20 years, the annuity costs were calculated to approximately NOK 460.000.

f) Increasing concrete cover

For calculating the appropriate concrete cover suitable for a service life of approximately 50 years, empirical models were used to calculate the time until both onset of corrosion and onset of cracking. The time in years prior to corrosion was empirically modelled by [14]:

$$t_{onset} = \frac{129 \cdot \left(\frac{S_i}{25.4}\right)^{1.22}}{K^{0.42} \cdot (w/c)} \tag{6}$$

where: S_i............ concrete cover [mm]
K............ chloride concentration [ppm]
w/c.......... water/cement ratio

For this particular structure in this particular environment, a concrete cover of 75 mm would give a time until onset of corrosion of approximately 25 years. The time until cracking was further calculated according to the empirical formulas by Liu [15]:

$$t_{cr} = \frac{Q_{cr}}{i_{cr}} \tag{7}$$

$$Q_{cr} = 0.602 \cdot d \cdot \left(1 + \frac{2 \cdot S_i}{d}\right)^{0.85} \tag{8}$$

where: d............. rebar diameter [mm]

i_{cr}............ corrosion rate [g/cm^2·day]

For a rebar diameter of 32 mm and an average corrosion rate of 0.5 μA/cm^2, the concrete cover would crack approximately one year after onset of corrosion. For the given structure, therefore, the first cracking of the beams would occur between 1990 and 1995. At this stage, it should be noted that an inspection report from 1991 did not reveal any visual damage, while serious cracking was observed a few years later in 1995. By increasing the concrete cover from 75 to 100 mm for the beams, however, the time until cracking would be be delayed so much that the owner's requirement of a service life 50 years would have been fulfilled. The additional material costs for increased concrete cover would be NOK 70.200 (58.5 m^3 concrete), which would give an increased total costs by 0.2 % to NOK 25.070.200. For an extended service life by approximately 10 years, the annuity costs were calculated to approximately NOK 500.000.

f) Increasing concrete quality

By increasing the concrete quality from 45 to 70 MPa for a Portland cement type of concrete, a durability design also indicates an increased service life by up to approximately 10 years. The material costs for the concrete would then be NOK 2.200.000. The increased costs of NOK 380.000 would increase the total costs by 1.5 % to NOK 25.380.000. For an assumed extended service life of approximately 10 years, the annuity costs were calculated to approximately NOK 500.000.

g) Increasing concrete quality and concrete cover

By combining the beneficial effects of increased concrete quality from 45 to 70 MPa and increased concrete cover from 75 to 100 mm in the beams, an estimated extended service life of approximately 25 years would have been obtained. The material costs for the concrete would then be NOK 2.420.000, giving an increased costs of NOK 580.000. This increased costs would increase the total costs by 2.3 % to NOK 25.580.000. For an assumed extended service life of approximately 25 years, the annuity costs were calculated to approximately NOK 380.000.

g) Cathodic protection

After 38 years in service, a cathodic protection system was installed in order to extend the service life of the beams by further 15 years. Since the cost of this installation was approximately NOK 3.000.000, the annuity costs for this situation were calculated to approximately NOK 540.000.

A summary of all the cost calculations for the various options are given in Table 2, from which it can be seen that a proper utilization of stainless steel would have given a very safe and cost-effective structure during the necessary service period.

Table 2. An overview of life cycle costs for various technical solutions and strategies.

	Additional service life [years]	$LCC(t_{end})$ [%]	Annuity costs $C_A(t_n)$ [x 10^6 NOK]
Doing nothing	± 0	100.0	0.63
Epoxy-coated rebars	+ 0	102.3	<0.64
Stainless steel rebars	>40	116.1	<0.38
75 % stainless steel rebars	>40	112.1	<0.35
Stainless steel clad rebars	>20	109.2	0.46
75 % stainless steel clad rebars	>20	106.9	0.46
Increased concrete cover	+ 10	100.2	0.50
Increased concrete quality	+ 10	101.5	0.50
Increased concrete cover and concrete quality	+ 25	102.3	0.38
Cathodic protection	+ 15	112.0	<0.54

3. Conclusions

For all concrete structures where safety and a reliable long-term performance are important, life cycle design including calculations of life cycle costs should be carried out. The results of the example presented in the present paper demonstrate that a proper utilization of stainless steel reinforcement would originally have been a very good investment and strategy for obtaining a good long term performance compared to that of the traditional design applied with the corresponding repair and maintenance costs.

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References

1. Gjørv O.E., "Durability and Service Life of Concrete Structures", *Proceedings, The first fib Congress 2002*, Session 8, Vol. 6, *Japan Prestressed Concrete Engineering Association*, Tokyo, 2002, pp. 1-16.
2. Takewaka, K. and Mastumoto, S., "Quality and Cover Thickness of Concrete based on the Estimation of Chloride Penetration in Marine Environments", ACI SP 109-17, *American Concrete Institute*, 1988, pp. 381-400.
3. "DuraCrete: Modelling of Degradation", The European Union - Brite EuRam III,

Project No. BE95-1347: "Probabilistic Performance Based Durability Design of Concrete Structures", Report No. R 4-5, 1998.

4. "DuraCrete: General Guidelines for Durability Design and Redesign", The European Union – Brite EuRam III, Project No. BE95-1347: "Probabilistic Performance Based Durability Design of Concrete Structures", Document R 15, February 2000, 109 p.
5. Gehlen, C. and Schiessl, P., "Probability-Based Durability Design for the Western Scheldt Tunnel", *Structural Concrete*, No. 2, 1999, pp. 1-7.
6. McGee, R., "Modelling of Durability Performance of Tasmanian Bridges", *Proceedings, The 8th International Conference on the Application of Statistics and Probability*, Sydney, Australia, 1999.
7. Raupach, M., "Chloride-Induced Macrocell Corrosion of Steel in Concrete – Theoretical Background and Practical Consequences", *Construction and Building Materials*, V. 10, 1996, pp. 329-338.
8. Pruckner, F., "Diagnosis and Protection of Corroding Steel in Concrete", Dr.ing. thesis 2002:140, *Norwegian University of Science and Technology, NTNU*, Trondheim, Norway, 2002, 199 p.
9. Vælitalo, S.H., Pruckner, F., Ødegård, O. and Gjørv. O.E., "A New Approach to Cathodic Protection of Corroding Concrete Harbor Structures", *Proceedings, Fourth International Conference on Concrete Structures under Severe Conditions: Environment and Loading*, Seoul, Korea, 2004.
10. Technical Data Bulletin, "Nuovinox 316L Clad rebar", *Stelax Industries Ltd*, Dallas, USA.
11. Griffith, A., Taylor, H.M., "Epoxy Coated Reinforcement Study", Final Report, *State Research Project # 527, Oregon Department of Transportation*, Oregon, USA, 1999.
12. Knudsen, A. and Skovsgaard, T., "The Optimal Use of Stainless Steel Reinforcement in Concrete Structures", *Proceedings, Third International Conference on Concrete under Severe Conditions - Environment and Loading, CONSEC '01*, Vancouver, Ed. N. Banthia, K. Sakai and O.E. Gjørv, V. 2, 2001,
13. Cramer, S.D., Covino Jr. B.S., Bullard, S.J., Holcomb, G.R., Russell, J.H., Nelson, F.J., Laylor, H.M. and Soltesz, S.M., "Corrosion Prevention and Remediation Strategies for Reinforced Concrete Coastal Bridges", *Cement & Concrete Research*, V. 24, 2002, pp.101-117.
14. Cheung, M.S. and Kyle, B.R., "Service Life Prediction of Concrete Structures by Reliable Analysis", *Construction and Building Materials*, V. 10, 1996, pp. 45-55.
15. Liu, Y., "Modeling the Time-to-Corrosion Cracking of the Cover Concrete in Chloride Contaminated Reinforced Concrete Structures", PhD thesis, *Virginia Polytechnic Institute and State University*, 1996, pp- 34-35.