Proceedings

Risk-Based Maintenance of Civil Structures

Delft, January 21st, 2003

Editorial board:
P.H.A.J.M van Gelder
A.C.W.M. Vrouwenvelder

Serie Workshop Proceedings no. 8

COLOFON

Serie Workshop Proceedings no. 8
Risk-Based Maintenance of Civil Structures

Faculty of Civil Engineering and Geosciences
Subfaculty of Civil Engineering
Section Hydraulic Engineering and Structural Mechanics
Delft University of Technology

Workshop:
Risk-Based Maintenance of Civil Structures
Proceedings of the workshop at the Delft University of Technology on January, 21st, 2003.
P.H.A.J.M. van Gelder, A.C.W.M. Vrouwenvelder
Section Hydraulic Engineering and Structural Mechanics
Faculty of Civil Engineering and Geosciences
ISBN 90-75729-10-3

Key words:
Risk-based Maintenance, Civil Structures

This publication is made by the
Research School Structural Engineering
P.O. Box 5048,
NL – 2600 GA DELFT
The Netherlands
Phone: (+31) (0)15 278 5833
Fax: (+31) (0)15 278 5767
E-mail: info@osbouw.nl
Internet: http://www.osbouw.nl

Section Hydraulic Engineering and Structural Mechanics, April 2004

Table of Contents

Preface

These proceedings contain the papers which were presented at a one-day workshop on Risk-Based Maintenance of Civil Structures at the Delft University of Technology on January, 21st, 2003. This workshop has been organised by the faculty of Civil Engineering and Geosciences of the Delft University of Technology with financial support from the Research School Structural Engineering. The workshop was based on the same principle as the previously organised workshop on Risk-Based Design of Civil Structures (Delft, January 11th, 2000): a 'public defence' of the jury members on PhD-committees (in 2000 Van Gelder's committee and in 2003 H.G. Voortman's committee). The papers at this workshop have been selected with strong relations to the risk-based maintenance of structures but on various terrains. Inspection plans (and its dependencies) for steel structures which are subject to fatigue are discussed as well as flood risk assessment and maintenance methods of water defense systems for coasts, rivers and estuaries. Furthermore, the workshop concentrated on deterioration and maintenance models of concrete structures with research results on chloride penetration of concrete structures and a case study on the North Sea environment. Finally, also work was presented on condition indicators for inspection and maintenance planning.

We express our thanks to the authors of the papers at this workshop (Prof. Faber, Prof. Oumeraci, Prof. Vrijling, Dr. Polder, Mrs. Li, Mrs. Malioka and Mr. Straub) for providing such interesting issues on the risk-based maintenance of civil structures, as well as the 40 participants who attended the workshop actively.

P.H.A.J.M van Gelder
A.C.W.M. Vrouwenvelder

Delft, December 15th 2003

Generic Inspection Plans for Steel Structures Subject to Fatigue

M. Faber

Swiss Federal Institute of Technology, ETH Zürich, Switzerland
e-mail: faber@ibk.baug.ethz.ch

Abstract

During the last 5 years significant efforts have been spend to develop practically applicable approaches for risk based inspection planning of steel structures subject to fatigue. These efforts have led to the development of the so-called generic inspection planning approach. Following this approach the fatigue sensitive details are categorized and described generically according to standard design parameters such as the Fatigue Design Factor (*FDF*), the SN curve, the hot spot stress field characteristics and the Reserve Strength Ratio (*RSR*). Due to the simplicity of the format of the developed inspection plans, the proposed approach has a high potential to be used in the general practical task of managing the integrity of welded steel structures but also as a basis for rules in codes for the design and maintenance of steel structures. The present paper summarizes some of the main constituents of generic risk based inspection planning and provides some references to the background literature where the interested reader will find more elaborate treatments of various specialist issues.

Introduction

Reliability and risk based inspection planning (RBI) for welded steel structures have been issues of high practical interest for especially the offshore engineering profession over the last two-three decades. The first initiatives and developments were directed toward inspection planning for welded connections subject to fatigue crack growth in fixed steel offshore structures and this particular application area for RBI is by now the most developed.

The first initiatives and developments were directed toward inspection planning for welded connections subject to fatigue in fixed steel offshore structures, see e.g. Skjong (1985) and Madsen et al. (1989). Later the same methodology was adapted to other structures such as e.g. tankers, FPSO's and semi-sub's see e.g. Soares and Gorbatov (1996) and Lotsberg et al. (1999). Structural reliability methods have played an important role in these developments.

Initially the practical implementation of the developed approaches required a significant amount and expertise in both the area of structural reliability theory and fatigue and fracture mechanics. Consequently the practical implementation of reliability and risk based inspection planning was lacking the impact on the industry it deserved.

Recently generic and simplified approaches for reliability and risk based inspection planning have been formulated, see Faber et al. (2000). These approaches facilitate a common generic modeling of fatigue sensitive structural details in terms of parameters, which are commonly applied in deterministic design of structures, such as the Fatigue Design Factor (*FDF*) and the Reserve Strength Ratio (*RSR*). Any engineer familiar with these basic design concepts may thus readily identify the inspection plans appropriate for a given detail in a given structure. The potential benefit for the practical implementation of reliability and risk based approaches to inspection and maintenance planning is significant. The generic inspection plans may also be applied as a decision tool for evaluating the effect of service life extensions, load increases and strengthening on the required inspection and maintenance effort. Finally generic inspection plans for welded connections can form a basis for codification of risk based inspection plans.

The present paper taking basis in the RBI developments in the offshore area summarizes some of the main constituents of generic risk based inspection planning. The presentation follows closely Faber et al. (2000), Faber (2003) and Faber et al. (2003) where the interested reader will find more detailed information.

The inspection and maintenance planning problem

Risk based inspection planning takes basis in the formulation of acceptance criteria specifying the acceptable risk in relation to safety to personnel, environmental risk and costs consequences. The inspection and maintenance activities are then planned such that the acceptance criteria are fulfilled throughout the service life of the considered engineering facility. The performance of engineering facilities i.e. the degradation over time is subject to a number of uncertainties. These include operational conditions, material characteristics and environmental exposure. The uncertainties have origin in inherent physical randomness and in uncertainties associated with the models used to assess the performance of the systems. If, furthermore, the statistical basis for the assessment of the uncertainties is limited then also statistical uncertainties may be important.

It is important to realise that the degree of control of the engineering systems achieved by the inspections is strongly influenced by the reliability of the inspections, i.e. their ability to detect and size degradation. The reliability and thus the information achieved by inspections are strongly dependent on the quality of the planned inspections, the coverage of the planned inspections and the times where the inspections are performed. The reliability of inspections themselves may be subject to very significant uncertainty and this must be taken into account in the planning of inspections and when the results of inspections are interpreted and used to update the predicted future performance of the considered engineering facility.

Given that inspections reveal a state of degradation, which is unacceptable, various methods of repairs may be implemented and the future performance of the engineering facility will thus depend on the choice of repair method as well as the quality of the implemented repair.

The problem complex is visualised in Figure 1.

Figure 1. Illustration of factors influencing Risk Based Inspection planning

From Figure 1 it is seen that the number of decision variables and dependencies underlying an inspection-planning problem is rather large and calls for a systematic treatment.

Requirements to safety and risk acceptance criteria

Requirements to the safety of offshore structures are commonly given in two ways. In the North Sea it is a requirement that the offshore operator demonstrates to the authorities that risk to personnel and risk to the

environment are controlled and maintained within acceptable limits throughout the operational service life of the installation. The specific limits are usually determined in agreement between the authorities and the offshore operator.
Normally, the requirements to the acceptable risk are given in terms of an acceptable Fatal Accident Rate (*FAR*) for the risk of personnel and in terms of acceptable frequencies of leaks and outlets of different categories for the risk to the environment. These acceptance criteria addresses in particular risk associated with the operation of the facilities on the topside and cannot be applied directly as a basis for the inspection planning of the structural components.
In addition to the general requirements stated above also indirect and direct specific requirements to the safety of structures and structural components are given in the codes of practice for the design of structures. As an example the NKB (1978) specifies a maximum annual probability of failure of 10^{-5} for structures with severe consequences of failure. For offshore structures no codes as of yet give specific requirements to the acceptable failure probability.
In regard to fatigue failures the requirements to safety are typically given in terms of a required Fatigue Design Factor (*FDF*). As an example NORSOK (1998) specifies the *FDF*'s specified in Table 1.

Table 1. Fatigue Design Factors

Classification of structural components based on damage consequence	Access for inspection and repair		
	No access or in the splash zone	Accessible Below splash zone	Accessible Above splash zone
Substantial consequences	10	3	2
Without substantial consequences	3	2	1

"Substantial consequences" in this context means that failure of the joint will entail:
- danger of loss of human life
- significant pollution
- major financial consequences.

By "Without substantial consequences" is understood failure, where it can be demonstrated that the structure satisfy the requirement to damaged condition according to the Accidental Limit States with failure in the actual joint as the defined damage.

From the *FDF*'s specified in Table 1 it is possible to establish the corresponding annual probabilities of failure. In principle the relationship between the *FDF* and the annual probability of failure has the form shown in Figure 2.

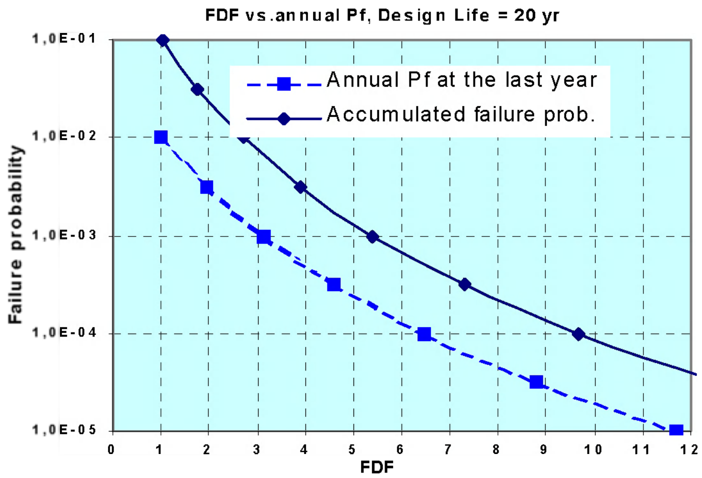

Figure 2. Principal relationship between *FDF* and probability of fatigue failure

For the joints to be considered in an inspection plan, the acceptance criteria for the annual probability of fatigue failure may be assessed through the Reserve Strength Ration (*RSR*) or the Residual Influence Factor (*RIF*) given failure of each of the individual joints to be considered together with the annual probability of joint fatigue failure.
If the *RSR* or the *RIF* given joint fatigue failure is known, it is possible to establish the corresponding annual collapse failure probability $P_{COL|FAT}$ if information is available on the applied characteristic values for the capacity, live load, wave height, ratio of the environmental load to the total load and coefficient of variation of the capacity.
The Residual Influence Factor (*RIF*) is defined as

$$RIF = \frac{RSR^{\text{damaged}}}{RSR^{\text{intact}}} \tag{1}$$

where RSR^{intact} is the RSR value for the intact structure and RSR^{damaged} is the RSR value for the structure damaged by fatigue failure of a joint.

The principal relation between the *RIF* and the annual collapse probability is illustrated in Figure 3.

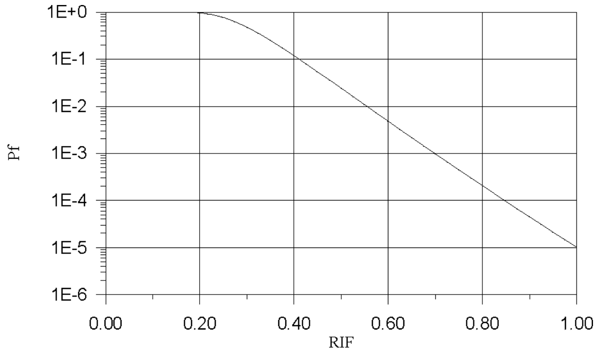

Figure 3. Example realtionship between Residual Influence Factors (*RIF*) and annual collapse probability of failure

The implicit code requirement to the safety of the structure in regard to total collapse may be assessed through the annual probability of joint fatigue failure. A typical maximal allowed annual probability of collapse failure is in the order of 10^{-3} - 10^{-5}.
On this basis it is possible to establish joint specific acceptance criteria in regard to fatigue failure. For each joint *j* the conditional probabilities of structural collapse give failure of the considered joint $P_{COL|FAT_j}$ are determined and the individual joint acceptance criteria for the annual probability of joint fatigue failure are found as

$$P_{AC_j} = \frac{P_{AC}}{P_{COL|FAT_j}} \tag{2}$$

The inspection plans must then satisfy that

$$P_{FAT_j} \leq P_{AC_j} \tag{3}$$

for all years during the operational life of the structure.
In addition to the acceptance criteria relating to the maximum allowable annual probabilities of joint fatigue failure economical considerations can be applied as basis for the inspection planning.

In addition to the acceptance criteria relating to the maximum allowable annual probabilities of joint fatigue failure economical considerations can be applied as basis for the inspection planning.
The aim is to plan inspections such that the overall service life costs are minimized. The costs include costs of failure, inspections, repairs and production losses. This will be considered in detail in the next.

The general inspection and maintenance problem

The performance of engineering systems over time is subject to a number of uncertainties. These include operational conditions, material characteristics and environmental exposure. The uncertainties have origin in inherent physical randomness and in uncertainties associated with the models used to assess the performance of the systems. If, furthermore, the statistical basis for the assessment of the uncertainties is limited then also statistical uncertainties may be important.

When inspection planning for engineering systems is considered, it is important to take all these uncertainties into consideration, as they will strongly influence the future performance of the systems. It is also important to realize that the degree of control of the engineering systems achieved by the inspections is strongly influenced by the reliability of the inspections, i.e. their ability to detect and size degradation. The reliability of inspections themselves may be subject to very significant uncertainty and this must be taken into account in the planning of inspections.

The decision problem of identifying the cost optimal inspection plan may be solved within the framework of pre-posterior analysis from the classical decision theory see e.g. Raiffa and Schlaifer (1961) and Benjamin and Cornell (1970). Here a short summary is given closely following Sørensen et al. (1991).

The inspection decision problem may be represented as shown in Figure 4.

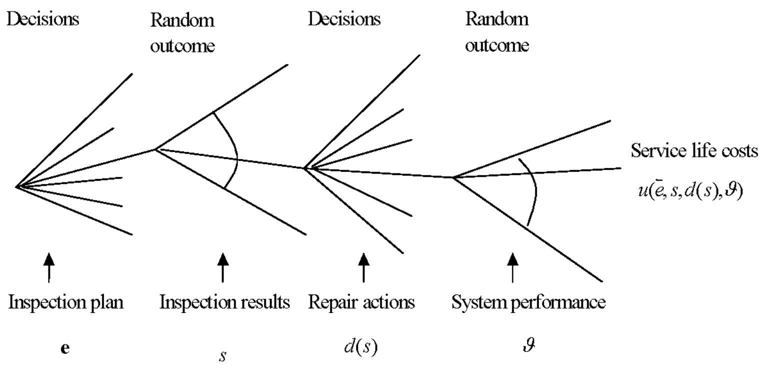

Figure 4. Inspection planning decision tree

In the general case the parameters defining the inspection plan are

- the possible repair actions i.e. the repair decision rule d
- the number of inspections N in the design lifetime T_L
- the time intervals between inspections $\mathbf{t} = (t_1, t_2, ..., t_N)^T$
- the inspection qualities $\mathbf{q} = (q_1, q_2, ..., q_N)^T$.

These inspection parameters are written as $\mathbf{e} = (N, \mathbf{t}, \mathbf{q})^T$. The outcome, typically a measured crack size, of an inspection is modelled by a random variable S. A decision rule d is then applied to the outcome of the inspection to decide whether or not repair should be performed. The different uncertain parameters (stochastic variables) modelling the state of nature such as load variables and material characteristics are collected in a vector $\mathbf{X} = (X_1, X_2, ..., X_n)^T$.

If the total expected costs are divided into inspection, repair, strengthening and failure costs and a constraint related to a maximum yearly (or accumulated) failure probability $\Delta P_F^{\max}$ is added then the optimization problem can be written

$$\begin{aligned} \min_{\mathbf{e},d} \quad & C_T(\mathbf{e},d) = C_{IN}(\mathbf{e},d) + C_{REP}(\mathbf{e},d) + C_F(\mathbf{e},d) \\ \text{s.t.} \quad & \Delta P_{F,t} \le \Delta P_F^{\max} \qquad t = 1,2,...,T_L \end{aligned} \tag{4}$$

where $C_T(\mathbf{e}, d)$ is the total expected cost in the design lifetime T_L, C_{IN} is the expected inspection cost, C_{REP} is the expected cost of repair and C_F is the expected failure cost. the annual probability of failure in year t is $\Delta P_{F,t}$. The N inspections are assumed performed at times $0 \le T_1 \le T_2 \le \ldots \le T_N \le T_L$.
If repair is assumed to be performed only when a crack is detected and has been measured to have a crack size a larger than a critical level a_r then the total number of repair realizations (branches) is 3^N see Figure 5.

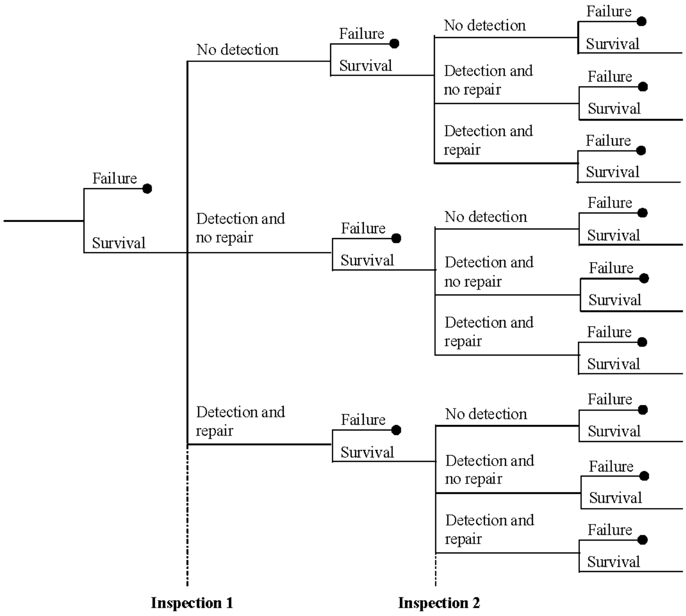

Figure 5. Repair, survival and failure event tree for a given inspection plan

It is noted that generally the total number of branches can be different from 3^N if the possibility of individual inspection times for each branch is taken into account.

The total capitalized expected inspection costs are

$$C_{IN}(\mathbf{e},d) = \sum_{i=1}^{N} C_{IN,i}(\mathbf{q})\big(1 - P_F(T_i)\big)\frac{1}{(1+r)^{T_i}} \tag{5}$$

The i th term represents the capitalized inspection costs at the i th inspection when failure has not occurred earlier, $C_{IN,i}(q_i)$ is the inspection cost of the i th inspection, $P_F(T_i)$ is the probability of failure in the time interval $[0, T_i]$ and r is the real rate of interest. The total capitalized expected repair costs are

$$C_{REP}(\mathbf{e},d) = \sum_{i=1}^{N} C_{R,i} P_{R_i} \frac{1}{(1+r)^{T_i}} \tag{6}$$

$C_{R,i}$ is the cost of a repair at the i th inspection and P_{R_i} is the probability of performing a repair after the i th inspection when failure has not occurred earlier and no earlier repair has been performed.
The total capitalized expected costs due to failure are estimated from

$$C_F(\mathbf{e},d) = \sum_{t=1}^{T_L} C_F(t)\Delta P_{F,t} P_{COL|FAT_j}(RSR)\frac{1}{(1+r)^{t}} \tag{7}$$

where $C_F(t)$ is the cost of failure at the time *t* and $P_{COL|FAT_j}(RSR)$ is the conditional probability of collapse of the structure given fatigue failure of the considered component *j*.
The probabilities of failure at year *t* may be determined from

$$\Delta P_{F_t} = P\left(\bigcup_k \left\{B_k(t-1) \cap S(t-1) \bigcap B_k(t) \cap \overline{S}^U(t)\right\}\right) \tag{8}$$

where $S(t)$ refers to the survival event for the component and $\overline{S}(t)$ to the failure event. $B_k(t)$ represents the different possible inspection and repair events leading up to the time *t*. $\overline{S}^U$ is the event of failure at time *t*. *U* indicates that the component characteristics may have changed due to repairs. The events over which the union is made are disjoint and the union may be substituted by a summation outside the probability operator.
The probability of repair may be found in a similar way by

$$P_R(t) = P(\bigcup_k \{S(t-1) \cap B_k(t-1) \cap R(t)\}) \tag{9}$$

where $R(t)$ is the repair event at inspection time *t*.
Details on the formulation of limit state equations for the modelling of failure, detection and repair events are given in Sørensen et al. (1991). Finally, the cumulative probability of failure at time T_i, $P_F(T_i)$ may be found by summation of the annual failure probabilities

$$P_F(T_i) = \sum_{t=1}^{T} \Delta P_{\,t} P_{COL|FAT_j} \tag{10}$$

Simplified and generic approaches

As an alternative to the so-called general approach outlined in the previous the parameters defining the inspection plan may be chosen as

- the possible repair actions i.e. the repair decision rule *d*
- the inspection qualities $\mathbf{q} = (q_1, q_2, ..., q_N)^T$
- the levels of acceptable annual probability of failure $\mathbf{P_f^T} = (P_{f1}^T, P_{f2}^T, ..., P_{fT_L}^T)^T$

If as before the total expected costs are divided into inspection, repair, strengthening and failure costs and a constraint related to a maximum yearly (or accumulated) failure probability $\Delta P_F^{\max}$ is added then the optimization problem can be written

$$\begin{aligned} &\min_{d,\mathbf{q},\mathbf{P_f^T}} \quad C_T(d,\mathbf{q},\mathbf{P_f^T}) = C_{IN}(d,\mathbf{q},\mathbf{P_f^T}) + C_{REP}(d,\mathbf{q},\mathbf{P_f^T}) + C_F(d,\mathbf{q},\mathbf{P_f^T}) \\ &\text{s.t.} \quad \Delta P_{Ft} \le P_{Ft}^T \qquad t = 1,2,...,T_L \end{aligned} \tag{11}$$

where C_T is the total expected cost in the service life T_L, C_{IN} is the expected inspection cost, C_{REP} is the expected cost of repair and C_F is the expected failure cost.
The solution of Equation (11) is in its general form just as difficult to obtain as the solution of Equation (4). However, if as an approximation it is assumed that all the components of $\mathbf{P_f^T} = (P_{f1}^T, P_{f2}^T, ..., P_{fT_L}^T)^T$ are identical (= P_f^T) the problem is greatly simplified. In this case Equation (11) may be solved in a practical manner by performing the optimization over P_f^T outside the optimization over *d* and q. The total expected

cost corresponding to an inspection plan evolving from a particular value of P_f^T is then evaluated over a range of values of P_f^T and the optimal $P_f^* = P_f^T$ is identified as the one yielding the lowest total costs.
In order to identify the inspection times corresponding to a particular P_f^T another approximation is introduced, namely that all the future inspections will result in no-detection. Thereby, the inspection times are identified as the times where the annual conditional probability (conditional on no-detection at previous inspections) of fatigue failure equals P_f^T. This is clearly a reasonable approximation for components with a high reliability.
Having identified the inspection times the expected costs are evaluated. It is important to note that the probabilities entering the cost evaluation are not conditioned on the assumed no-detection at the inspection times. This in order to include all possible contributions to the failure and repair costs.
The process is repeated for a range of different values of P_f^T and the value P_f^*, which minimizes the costs and at the same time fulfils the given requirements to the maximum acceptable P_f^T is selected as the optimal one.
The optimal inspection plan is then the inspection times $0 \leq T_1 \leq T_2 \leq ... \leq T_N \leq T_L$ corresponding to P_f^*, the corresponding optimal repair decision rule *d* and inspection qualities q. Based on the approach outlined in the above it is now possible to establish so-called generic inspection plans.
Following the approach outlined above it is possible to establish so-called generic inspection plans. The idea is to establish pre-fabricated inspection plans for different joint types designed for different fatigue lives.

For given
- type of fatigue sensitive detail – and thereby code-based SN-curve
- fatigue strength measured by *FDF* (Fatigue Design Factor)
- importance of the considered detail for the ultimate capacity of the structure, measured e.g. *RSR* of *RIF*
- inspection, repair and failure costs

the optimal inspection plan i.e. the inspection times, the inspection qualities and the repair criteria, can be determined. This inspection plan is generic in the sense that it is representative for the given characteristics of the considered detail, i.e. SN-curve, *FDF*, *RSR* and the inspection, repair and failure costs.

For given SN-curve, *FDF* and cost structure the procedure may be summarized as follows.
1. Identify inspection times by assuming inspections at times when the annual failure probability exceed a certain threshold.
2. Calculate the probabilities of repairs corresponding to the times of inspections
3. Calculate the total expected costs.
4. Repeat step 1-3 for a range of different threshold values and identify the optimal threshold value as the threshold value yielding the minimum total costs.

The inspection times corresponding to the optimal threshold value then represents the optimal inspection plan. For the identification of optimal inspection methods and repair strategies the above mentioned procedure may be looped over different choices of these. The procedure is illustrated in Figure 6.

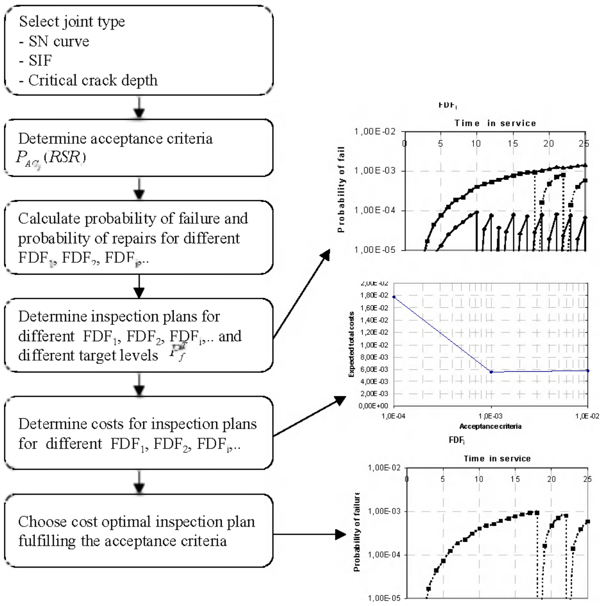

Figure 6. Illustration of the flow of the generic inspection planning approach

As the generic inspection plans are calculated for different values of the *FDF* it is possible directly to assess the effect of design changes, the effect of strengthening of joints and/or extensions in the service life on existing structures as such changes are directly represented in changes of the *FDF*.
Furthermore it is also interesting to observe that the effect of service life extensions on the required inspection efforts may be directly assessed through the corresponding change on the *FDF*. Given the required service life extension the *FDF* for the joint is recalculated and the corresponding pre-fabricated inspection plan identified.

Probabilistic fatigue modelling

In the following the probabilistic models for fatigue assessment based on SN-curves and fracture mechanics are briefly summarized.

Assessment of SN fatigue lives

It is assumed that the SN-curve is bilinear:

$$N = K_1 \left(\frac{\Delta s}{(T / T_{ref})^{\alpha^*}} \right)^{-m_1} \quad \text{for } N \leq N_C \tag{12}$$

$$N = K_2 \left(\frac{\Delta s}{(T / T_{ref})^{\alpha^*}} \right)^{-m_2} \quad \text{for } N > N_C \tag{13}$$

where Δs : stress range, N : number of cycles to failure, K_1, m_1 : material parameters for $N \leq N_C$, K_2, m_2: material parameters for $N > N_C$, Δs_C : stress range corresponding to N_C, T : thickness, T_{ref} : reference thickness and α^* : scale exponent.
Further it is assumed that the total number of stress ranges for a given fatigue critical detail can be grouped in n_σ groups / intervals such that the number of stress ranges in group *i* is n_i per year.
The code-based design equation is written:

$$G = 1 - \sum_{s_i \geq \Delta s_C} \frac{n_i T_F}{K_1^C s_i^{-m_1}} - \sum_{s_i < \Delta s_C} \frac{n_i T_F}{K_2^C s_i^{-m_2}} = 0 \tag{14}$$

where

$s_i = \dfrac{Q_i}{z} \dfrac{1}{(T/T_{ref})^{\alpha^*}} = \dfrac{Q_i}{z^*}$ stress range in group i

Q_i action effect (proportional to stress range s_i in group i

z design parameter

z^* modified design parameter taking into account thickness effects

K_i^C characteristic value of K_i (mean of $\log K_i$ minus two standard deviations of $\log K_i$)

T_F fatigue life time

The design parameter z^* is determined from the design Equation (14). Next, the reliability index (or the probability of failure) is calculated using this design value and the limit state function associated with Equation (14). The coefficient of variation COV_{wave} models the uncertainty on the wave load, foundation stiffness and stress ranges. COV_{SCF} models the uncertainty in the stress concentration factors (SCF) and local joint flexibilities. The limit state equation can be written:

$$g = 1 - \sum_{s_i \geq \Delta s_C} \frac{n_i T_L}{K_1 s_i^{-m_1}} - \sum_{s_i < \Delta s_C} \frac{n_i T_L}{K_2 s_i^{-m_2}} \tag{15}$$

where

$s_i = X_S \dfrac{Q_i}{z^*}$ stress range in group i

X_S stochastic variable modeling model uncertainty related to waves and SCF. X_S is Log-Normal distributed with mean value = 1 and $COV = \sqrt{COV_{wave}^2 + COV_{SCF}^2}$

K_i $\log K_i$ is modeled by a Normal distributed stochastic variable according to a specific SN-curve.

T_L service life

Using the stochastic model in Table 2 and Equation (15) the probability of failure in the service life and the annual probability of failure can be obtained.

Table 2. Stochastic model

Variable	Distribution	Expected value	Standard deviation
Z_{SCF}	LN	1	COV_{SCF} =0.00 / 0.05 / 0.10 (Curve 1) COV_{SCF} =0.15 / 0.20 (Curve 2)
Z_{vawe}	LN	1	COV_{wave} =0.10 / 0.15
T_F	D	25 – 400 years	
T_L	D	T_F / FDF	
m_1	D	3	
$\log K_1$	N	12.048 (F) 12.713 (T)	0.218 0.200
m_2	D	4	
$\log K_2$	N	13.980 (F) 14.867 (T)	0.291 0.267
$\log K_1$ and $\log K_2$ are assumed fully correlated			

Assessment of FM fatigue lives

A fracture mechanical modeling of the crack growth is applied assuming that the crack can be modeled by a 2-dimensional semi-elliptical crack. It is assumed that the fatigue life may be represented by a fatigue initiation life and a fatigue propagation life.
The following model is used:

$$N = N_I + N_P \tag{16}$$

where N : number of stress cycles to failure, N_I : number of stress cycles to crack propagation and N_P : number of stress cycles from initiation to crack through.
The number of stress cycles from initiation to crack through is determined on the basis of a two-dimensional crack growth model. The crack is assumed to be semi-elliptical with length $2c$ and depth a, see Figure 7.

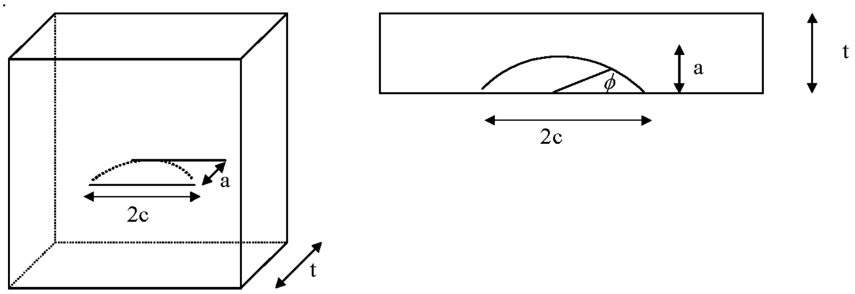

Figure 7. Semi-elliptical surface crack in a plate under tension or bending loads

The crack growth can be described by the following two coupled differential equations.

$$\begin{aligned} \frac{da}{dN} &= C_A(\Delta K_A)^m \qquad a(N_0) = a_0 \\ \frac{dc}{dN} &= C_C(\Delta K_C)^m \qquad c(N_I) = c_0 \end{aligned} \tag{17}$$

where C_A, C_C and m are material parameters, a_0 and c_0 describes the crack depth *a* and crack length *c*, respectively, after N_I cycles and where the stress intensity ranges are ΔK_A and ΔK_C. ΔK_A and ΔK_C are obtained based on the models in Newmann & Raju (1981) and Smith & Hurworth (1981).

The sum of the membrane stresses, σ_t and the bending stresses, σ_b is taken as

$$\sigma_t + \sigma_b = \Delta\sigma \tag{18}$$

It is assumed that the ratio between bending and membrane stresses is η implying that

$$\sigma_t = \frac{1}{\eta+1}\Delta\sigma \quad \text{and } \sigma_b = \frac{\eta}{\eta+1}\Delta\sigma \tag{19}$$

Load shedding (linear moment release) is introduced as described in Aaghaakouchak et al. (1981).
The stress range $\Delta\sigma$ is obtained from

$$\Delta\sigma = Z_{wave} Z_{SCF} \Delta\sigma^e \tag{20}$$

where

Z_{wave} and Z_{SCF} model uncertainties

$\Delta\sigma^e$ equivalent stress range: $$\Delta\sigma^e = \left[\frac{1}{n}\sum_{i=1}^{n_\sigma} n_i \Delta\sigma_i^m\right]^{1/m} \tag{21}$$

The total number of stress ranges per year, n is

$$n = \sum_{i=1}^{n_\sigma} n_i \tag{22}$$

In the assessment of the equivalent constant stress range the effect of a possible lower threshold value ΔK_{TH} on the crack growth inducing stress intensity factor ΔK has not been taken directly into account. This effect is assumed implicitly accounted for by evaluation of Equation (21) using the appropriate SN-curve exponent *m*.

The crack initiation time N_I is modeled as Weibull distributed with expected value μ_0 and coefficient of variation equal to 0.35, see e.g. Lassen (1997).

The limit state function is written

$$g(\mathbf{x}) = N - nt \tag{23}$$

where t is time in the interval from 0 to the service life T_L.

In order to model the effect of different weld qualities two different values of the crack depth at initiation a_0 are used: 0.1 mm and 0.5 mm corresponding approximately to high and low material control. The corresponding assumed length c_0 is 5 times the crack depth. The critical crack depth a_c is taken as the thickness of the tubular. The probabilistic modeling used in the fracture mechanical reliability analysis is shown in Table 3.

The parameters $\mu_{\ln C_C}$ and μ_0 are now fitted such that the probability distribution function for the fatigue lives determined using the SN-approach and the fracture mechanical approach are as identical as possible.

Table 3. Uncertainty modeling used in the fracture mechanical reliability analysis.

D: Deterministic, N: Normal, LN: LogNormal, W: Weibull.

Variable	Dist.	Expected value	Standard deviation
N_I	W	μ_0 (fitted)	0.35 μ_0
a_0	D	0.1 mm (high material control) / 0.5 mm (low material control)	
$\ln C_C$	N	$\mu_{\ln C_C}$ (fitted)	0.77
m	D	m-value corresponding to the low cycle part of the bi-linear SN-curve	
Z_{SCF}	LN	1	0 / 0.05 / … / 0.20
Z_{vawe}	LN	1	0.10 / 0.15
n	D	Total number of stress ranges per year	
a_c	D	T (thickness)	
η	D	2 / 4	
Y	LN	1	0.1
T	D	10 mm/ 30 mm / 50 mm / 100 mm	
T_L	D	20 years / 25 years	
T_F	D	$= FDF\ T_L$ = 25 / 50 / … / 250 years	
$\ln C$ and N_0 are correlated with correlation coefficient $\rho_{\ln(C),N_0} = -0.50$			

Example

A steel jacket structure installed in year 2000 with service life T_L=40 years and located in the southern part of the North Sea is considered. The characteristics for a representative selection of fatigue sensitive details are shown in Table 4.

Table 4. Example cases

Case	COV_{wave}	COV_{SCF}	SN-curve	T [mm]	T_F [year]	DoB
1	0.1	0.15	T	50	100	0.6
2	0.1	0.10	T	50	100	0.6
3	0.1	0.15	T	30	100	0.6
4	0.1	0.15	T	50	200	0.6
5	0.1	0.15	T	50	100	0.3
6	0.1	0.10	F	50	100	0.6
7	0.1	0.05	F	50	100	0.6
8	0.1	0.10	F	30	100	0.6
9	0.1	0.10	F	50	200	0.6
10	0.1	0.10	F	50	100	0.3

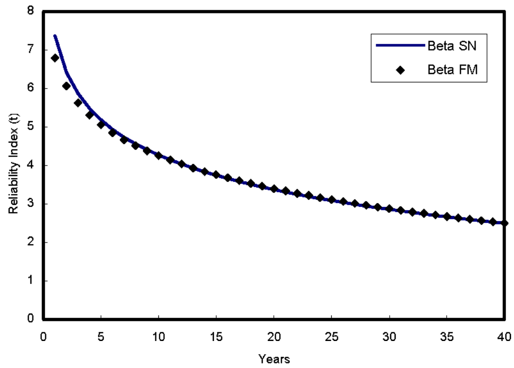

Figure 8. Reliability indices for SN and calibrated Fracture Mechanics corresponding to accumulated probability of failure.

In Figure 8 the reliability indices (corresponding to accumulated probabilities) for the limit states based the SN-approach and the calibrated fracture mechanics (FM) approach are illustrated for case 1 (see Table 4). It is seen that a very good correspondence is obtained between the two different approaches.

In Figure 9 the resulting inspection plans obtained by iPlan are shown corresponding to a maximum acceptable annual probability of failure equal to $\Delta P_F^{\max} = 10^{-5}$. It is seen that COV_{SCF} and the fatigue life T_F are important for the inspection plan. Reducing the uncertainty of the stress ranges or extending the fatigue life increase the year of the first inspection and can reduce the number of inspections.

Project
OMAE'03 Example

Date: 2003-01-15

Prepared by: JDS
Checked by: MHF
Approved by: DMS

Inspection plans according to the user-defined thresholds:

Index **Inspections**

Year	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036	2037
1														X						X								X										
2																			X													X						
3														X						X						X									X			
4														X					X						X									X				
5																	X								X										X			
6																			X													X						
7																										X												
8																			X											X								
9																			X											X								
10																								X														

Figure 9. Inspection plan obtained by iPlan

Conclusions

The present paper outlines recent developments on simplified and generic approaches to risk and reliability based inspection and maintenance planning. Acceptance criteria have been considered for the inspection planning of fatigue sensitive structural details of steel jacket structures. It has been described how the acceptable annual probability of fatigue failure for specific details may be derived on the basis of the *RSR* or the *RIF* for the structure given fatigue failure of the considered detail.

It is described how generic inspection plans can be derived for different structural details (SN curves), *FDF* values, *RSR* or *RIF* values, inspection, repair and failure costs and inspection / repair methods.

Based on the generic and simplified approach for risk based inspection planning a data-base iPlan has been developed and it is illustrated how inspection plans may be obtained from interpolation between the pre-defined generic plans.
The generic inspection plans may also be applied as a decision tool for evaluating the effect of service life extensions, load increases and strengthening on the required inspection and maintenance effort. The format of the generic inspection plans appears to be appropriate for codification in regard to inspection effort of fatigue sensitive details.

References

Aaghaakouchak, A., Glinka, G. and Dharmavasan, S. (1989*). A load shedding model for fracture mechanics analysis of fatigue cracks in tubular joints*. Proc. OMAE'89, The Hague, pp. 159-165.

Benjamin, J.R. and Cornell, C.A. (1970). *Probability, statistics and decision for civil engineers*. McGraw-Hill, NY.

Faber, M.H., Engelund, S., Sørensen, J.D. & Bloch, A. (2000*). Simplified and generic risk based inspection planning*. Proc. OMAE 2000, S&R paper 6143.

Faber, M.H. (2003). *New Approaches to Inspection Planning of Fatigue Damaged Offshore Platforms.* Journal of UK Safety and Reliability Society, Volume 23, No.1, Winter 2002/3, pp. 35-50 (ISSN 0960-7353).

Faber, M.H., Sørensen, J.D., Tycksen, J. and Straub, D. (2003). *Field Implementation of RBI for Jacket Structures.* Proceedings to the International Conference on Offshore Mechanics and Arctic Engineering in Cancun, Mexico. In press.

Lassen, T. (1997). *Experimental investigation and stochastic modeling of the fatigue behaviour of welded steel joints*. PhD thesis, Structural reliability theory, paper No. 182, Aalborg University.

Lotsberg, I. and Sigurdsson, G., Wold, P.T., (1999), *Probabilistic inspection planning of the Åsgaard A FPSO hull structure with respect to fatigue*. Proc. OMAE99, New Foundland, Canada.

Madsen, H.O., Sørensen, J.D. Olesen, R., (1989), *Optimal inspection planning for fatigue damage of offshore structures*, In Proc. ICOSSAR 89, San Francisco, USA, pp. 2099-2106.

Newmann, J. and Raju, I. (1981). *An empirical stress-intensity factor for the surface crack.* Engineering Fracture Mechanics, Vol. 22, No. 6, pp. 185-192.

Nordic Committee on Building Regulations, NKB, (1978). *Recommendations for loading and safety regulations for structural design*. NKB-Report, No. 36, Copenhagen.

NORSOK (1998). *Design of Steel Structures N-004*, Rev. 1, December 1998.

Raiffa, H. and Schlaifer, R. (1961). *Applied Statistical Decision Theory*. Harward University Press, Cambridge, Mass.

Skjong, R., (1985), *Reliability based optimization of inspection strategies*. In Proc. ICOSSAR 85 Vol. III, Kobe, Japan, pp. 614-618.

Smith, I.J. and Hurworth S.J. (1981). *Probabilistic fracture mechanic evaluation of fatigue from weld defects in butt welded joints*. Proc. Fitness for purpose validation of welded constructions, London.

Soares, C. G. and Gorbatov, Y. (1996), *Fatigue Reliability of the Ship Hull Girder Accounting for Inspection and Repair*, Journal of Reliability Engineering and Systems Safety, 51:2, 1996, pp. 341-351.

Sørensen, J.D., Faber, M.H., Rackwitz, R. and Thoft-Christensen, P.T. (1991). *Modeling in optimal inspection and repair*. Proc. OMAE'91, Stavanger, Norway, pp.281-288.

Flood Risk Assessment and Mitigation for Coasts and Estuaries

H. Oumeraci

Leichtweiss-Institut (LWI), TU Braunschweig and Joint Research Centre for Coastal Engineering (FZK) of the University of Hannover and the Technical University of Braunschweig
e-mail: h.oumeraci@tu-bs.de

Abstract

A conceptual framework based on probabilistic risk analysis (PRA) is proposed for the design of coastal flood defences which meets the sustainability requirements. The overall framework includes the management of the remaining risk as an integral part of the design process. The implementation of the risk analysis requires (i) the prediction of the flood risk, (ii) the evaluation of the acceptable flood risk and (iii) the evaluation of the flood risk level which is obtained through comparison of the predicted and acceptable flood risk.

Necessity of new design approach for coastal flood defences

Coastal flood defence has a long tradition worldwide. In spite of the variety of design methods and safety standards adopted in each country, the design criteria for flood defence structures are still essentially based on design water levels associated with specific exceedance frequencies. This is exemplary shown in Figure 1 for the design of sea dikes as it is presently practised in Northern Europe.
The specified exceedance frequency is implicitly interpreted as a failure probability which is again equated to a flooding probability. This approach is too simplistic, as it may lead for instance to:

1. too high and expensive dikes, because the dike must not necessarily fail when the design water level is exceeded (modern dikes have generally a substantial safety margin!), so that the catastrophic water level might certainly be much higher than the design water level
2. an incorrect safety assessment, because the dike may also fail, even if the design water level will not be exceeded (seaward slope and toe failure, piping etc.), thus leading to a dike breach and subsequent devastating damages in the protected area.

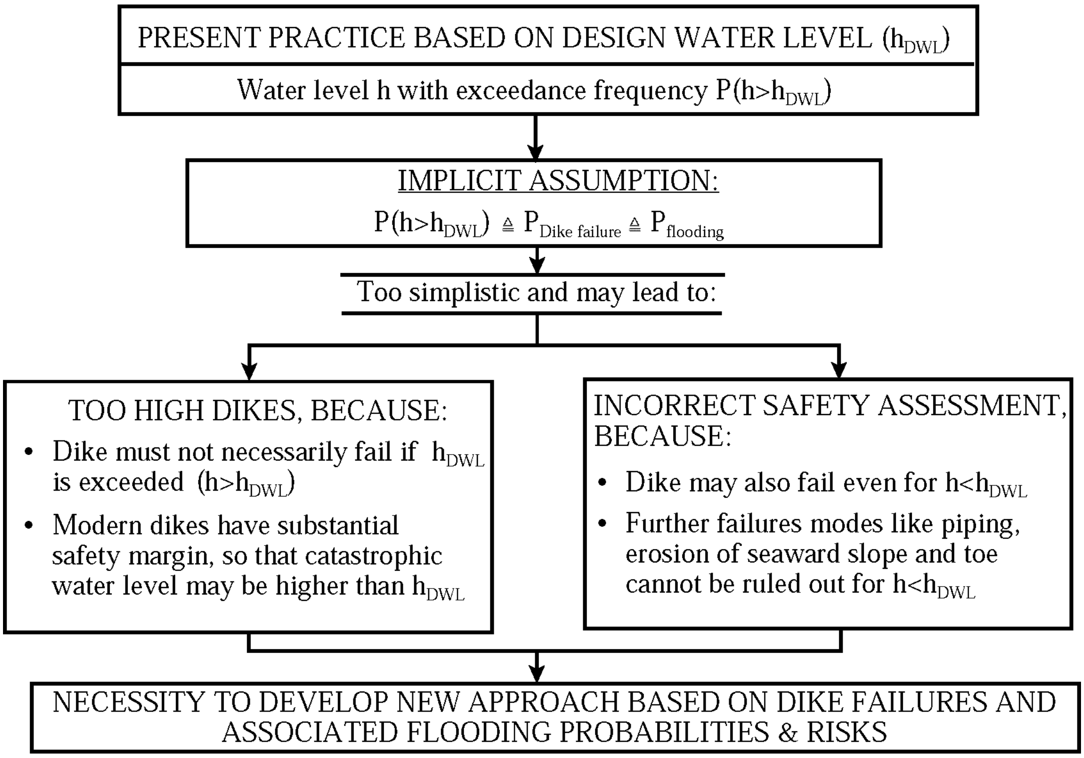

Figure 1. Present design practice for coastal flood defences and necessity of new approach

Moreover, the present design practice is not only inconsistent with sustainable flood protection of coastal zones, but is also lacking rationality and transparency which constitute both an indispensable basis for a wide acceptability and thus for the unification and harmonisation of safety standards of coastal flood defences.

These and further considerations suggest that a design approach - based on the failure probability of flood defence structures, associated flood probabilities and risks - must be developed.

Among the reasons pleading for a probabilistic risk analysis (PRA) as the sole candidate framework through which rational, transparent and thus widely accepted and harmonised safety criteria can be achieved, the following are noteworthy: (i) the large variety of the involved aspects, together with their uncertainties which have to be addressed explicitly in the analyses, (ii) the integrated nature and the high complexity of the design problem, as well as (iii) the necessity to harmonise design and safety standards in various fields (coastal engineering, dam engineering transportation, nuclear power plants, etc.). Innovative results within such a PRA framework are not only expected with respect to the overall risk analysis procedure for coastal flood defences, but also with respect to the related prospective models, techniques and methodologies. Noteworthy in this respect are among others the models to predict the following processes and issues: (i) wave transformation on shallow foreshores with complex topographies, including the joint probability of water levels and wave parameters as well as the associated uncertainties; (ii) failure mechanisms of coastal flood defence structures, their interaction and consequences on the flooding probability; (iii) breaching mechanisms as well as the subsequent flood wave propagation and potential damages; (iv) acceptable flood risks within the protected areas by accounting for economic losses, loss of life and further intangibles like environmental and cultural losses.

Moreover, the new direction forward should provide a detailed scientific and technical integrated framework which will (i) explicitly address the uncertainties through a comprehensive reliability based approach, (ii) help to bridge the gap between technical and non technical decision makers through the introduction of the risk concept and a new risk scale and (iii) build a sound basis for a broader and a more general framework for the management of coastal flood risks, including strategies for monitoring, inspection, maintenance, repair, review and safety evaluation updates as well as for emergency measures.

The new conceptual PRA-based framework as a response to the sustainability challenge: a brief outline

A recently completed MAST III-Project on "Probabilistic Design Tools of Vertical Breakwaters (PROVERBS)" which was led by the author, concluded that *"the design process for coastal structures is expected to develop within the next decade from pure deterministic to probabilistic analysis methods embedded into a risk based design and risk management framework to achieve sustainable protection of the coastal zones"* (Oumeraci et al. 2000a). Based on the results of PROVERBS and the lessons drawn from this and other projects on coastal defences, the conceptual PRA-based framework shown in Figure 2 has been developed for the design of coastal flood defences: (i) prediction of flood risk, (ii) evaluation of acceptable flood risk, (iii) evaluation of the remaining risk/risk level through comparison of predicted and acceptable risk and (iv) management of the remaining risk. One of the key features of this design framework is the incorporation of the risk management as an integral part of the design process. In fact, no design optimisation would be possible without the knowledge of the remaining risk and its management (Figure 2).

The main sources and types of uncertainties which must be explicitly considered in the PRA framework are summarised in Figure 3. Some methods on how to assess and consider these uncertainties in PRA have been used for vertical breakwaters (Oumeraci et al. 2000a), but further sophisticated methods such as fuzzy sets, elicitation of expert opinions etc. are getting more and more operational and must also be applied (Cooke 1991).

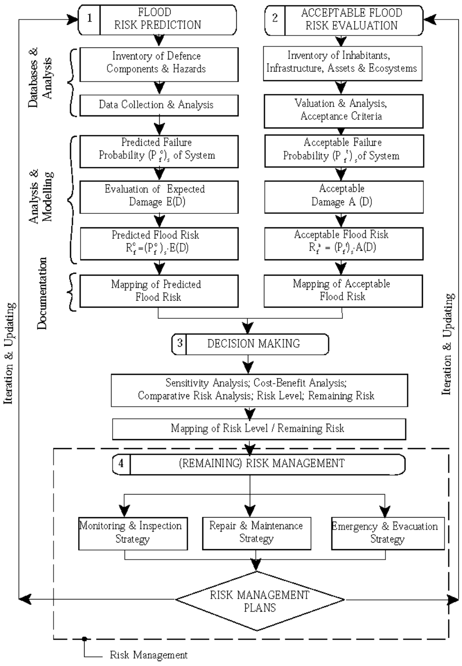

Figure 2. Probabilistic Risk Analysis Based framework fot the design of coastal defences

In the following sections particular focus will be put on the methodologies related to three aspects illustrated by Figure 4, including (i) the prediction of flood risk, (ii) the evaluation of acceptable risk and (iii) the calculation of the flood risk level (remaining risk). In view of the constraints associated with the limited length of the paper, the fourth aspect (management of remaining risk in Figure 2) could be addressed in a forthcoming paper.

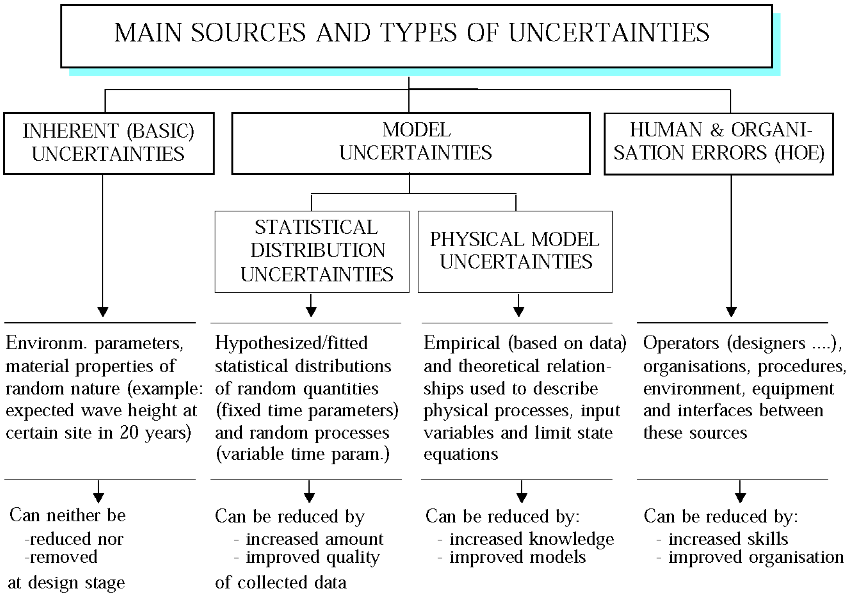

Figure 3. Uncertainties to be considered in proposed PRA-based framework (adopted from Oumeraci et al. 2000)

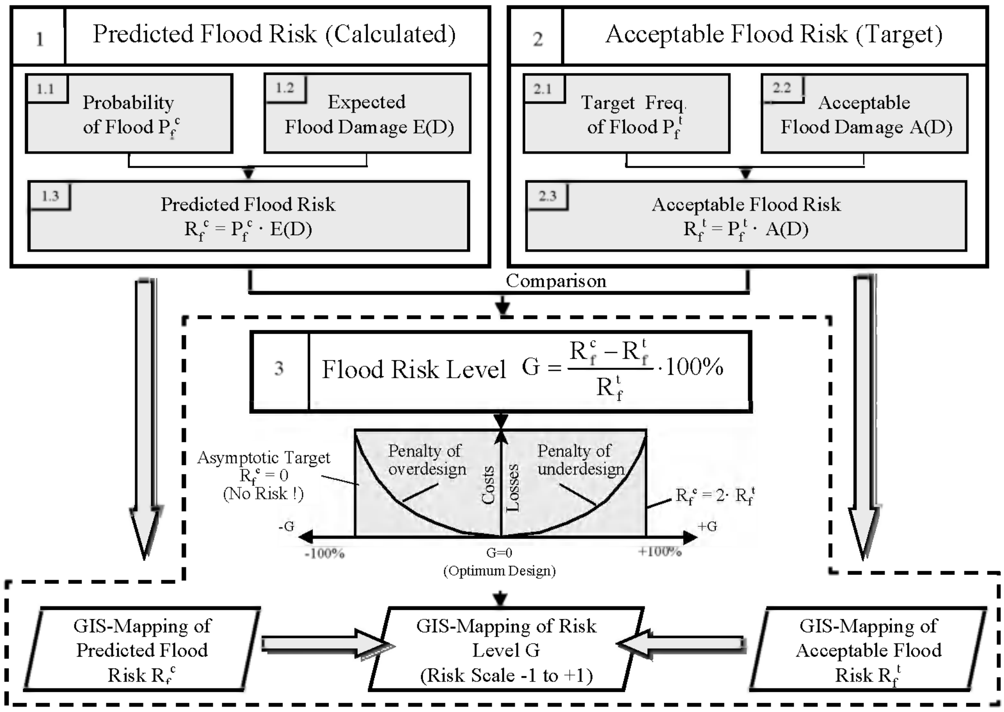

Figure 4. Methodology for the assessment of the remaining risk

Prediction of flood risk

The prediction of the flood risk requires the knowledge and associated uncertainties of (i) the morphological, topographic, hydraulic and other boundary conditions, (ii) the failure modes of the defence components, their interactions and related limit state equations and (iii) the breaching of the defence structures as well as the flood wave propagation and the subsequent damages which would result in the protected area.

Topographic, hydraulic and further boundary conditions

First, the flood defence scheme, including the foreshore topography, the entire chain of flood defence structures must be described, together with the protected areas, facilities and infrastructures (socio-economic aspects). The description must be performed at different scales and levels of detail, depending on the purpose under consideration. Basically, both a cross sectional representation (Figure 5) and a plan view representation (Figure 6) are needed. The former is particularly important for the analysis of the hydraulic boundary conditions (water levels and waves) and the effect of the interaction between the various failures of the components (high foreshores, dikes, dunes etc.) of the defence chain on the flooding probability. The plan view representation is relevant for the analysis of the overall failure of defence components (spatial correlation), the subsequent flood wave propagation and its damaging effects in the protected area (see Figure 11).

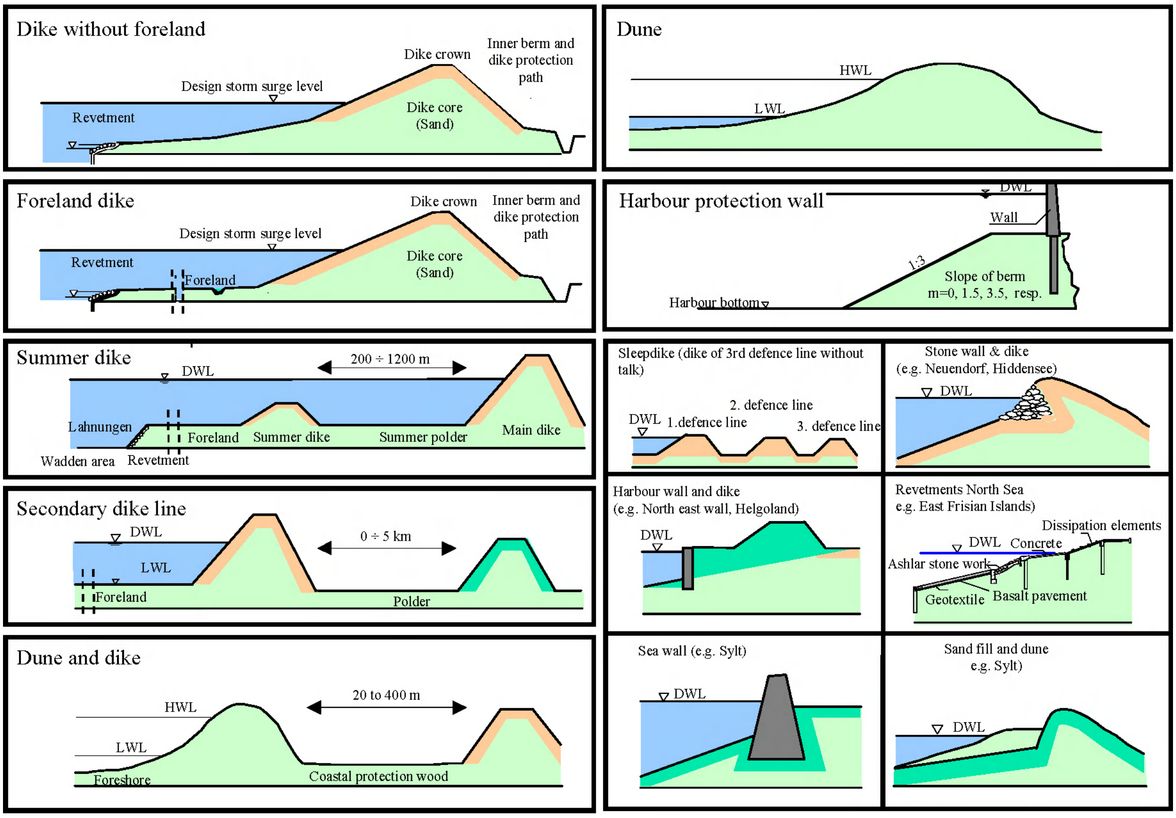

Figure 5. Coastal flood defence chain (cross-sectional representation)

From the view point of safety and risk classes, some fundamental cases must be distinguished. Depending on the source of the hazards there are two typical cases: (i) threat from both sea and river (Figure 6a) and (ii) threat only from the sea (Figure 6b). Depending on the conditions in the protected areas, typical situations with short or long propagation time of the flood wave as well as situations with high and low urbanisation level may be encountered, thus requiring different scales and detail levels of description and mapping (GIS).
Second, the hydraulic boundary conditions must be reliably assessed. This particularly includes (i) the joint probability of water levels and waves and (ii) transformations of waves propagating over the shallow foreshore to obtain the design waves at the defence structures. In fact, both water levels and associated wave conditions at the structure belong to the input parameters which are vital for any design. Small errors in these inputs may lead to much larger errors for outputs such as wave loads, overtopping and structure

stability. One of the key findings of the EU/MAST III project PROVERBS on „Probabilistic Design Tools for Vertical Breakwaters“ (Oumeraci et al. 2000a) was that (i) the uncertainties of the wave loads still represent the major uncertainty in the entire design process and (ii) these uncertainties essentially originate from the errors in predicting wave transformation from deep water towards and over shallow foreshores. However, the most important source of uncertainty is due to the lack of knowledge and appropriate data on the joint probability of water levels and waves.
For this reason and because the joint occurrence of water level and waves provide the input data required for the prediction of wave transformation propagating into shallow foreshores, the problems associated with the joint probability of water levels and waves, including some indications on future research, are first discussed before addressing the problems associated with wave transformation and the uncertainties in predicting waves over shallow foreshores.

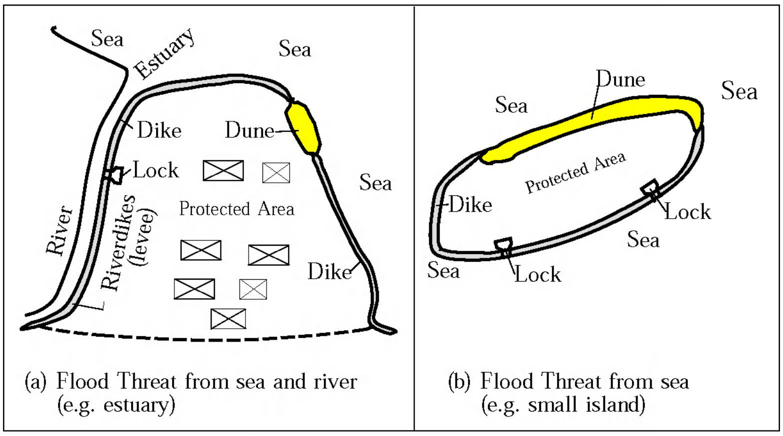

Figure 6. Coastal flood defence scheme (plan view representation)

Joint probability of storm water levels and waves

Disastrous damages to sea defences are often caused by unfavourable combinations of water levels and waves during storms. Therefore, the development of more appropriate and practical approaches to predict such extreme conditions becomes a key issue in any PRA-based design of coastal flood protection. To obtain homogeneous data sets for water levels it is essential to distinguish between (i) astronomical tidal components which are deterministic and which may change due to human interference (dredging, closure of estuaries etc.), and (ii) the meteorological forcing components which represent the stochastic surge part of the actually measured water levels (Figure 7).

The yet available attempts to describe the joint probability of extreme water levels and waves do not explicitly include the distribution of wave periods (Figure 8). In some circumstances however, wave periods can be as important as wave heights in predicting structure responses such as wave overtopping, especially when waves are limited by depth.

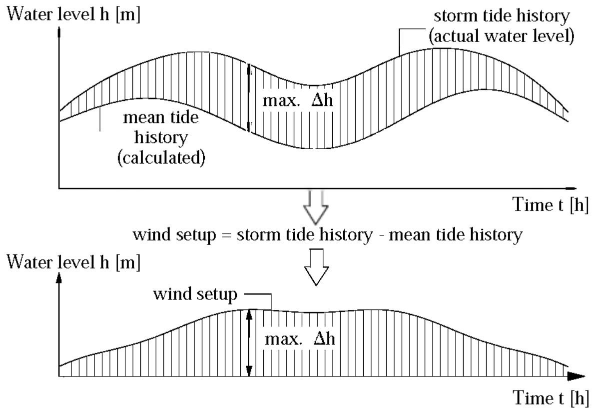

Figure 7. Surge and astronomical tidal levels (principle sketch)

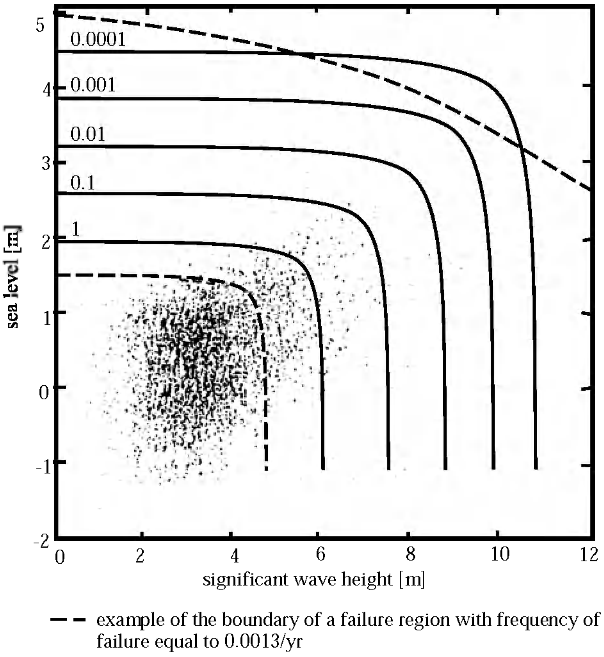

Figure 8. Joint distribution of extreme water levels and waves (De Ronde et al. 1995)

Therefore, future approaches must explicitly include the variability of wave periods. The joint dependence between wave heights and periods can also be obtained by considering the variability of wave steepness which may represent a more robust variable than the wave period for statistical calculation.

Moreover, the future prediction methods should also enable (i) an explicit consideration of additional non-simultaneous data and information, (ii) an easy assessment of uncertainties and of their combined effect on the result, (iii) a long-term simulation to produce extreme values of water levels, of wave heights with their associated periods and of their combination. Research towards the development of such methods is underway (e.g. Owen et al. 1997). Where possible, uncertainties should be assessed from statistical data. Otherwise, elicitation of expert opinions may represent a reasonable alternative (Cooke 1991).

Uncertainties in predicting waves over shallow foreshores

Coastal defences are generally attacked by waves which have propagated over shallow foreshores with complex morphological features before reaching the main defence line. Therefore, the waves approaching the defence line are subject to a variety of transformation processes including depth-limited wave breaking, wave reformation, etc. These processes and the subsequent changes in the wave height distribution have to be simulated in order to obtain the distribution just in front of the defence line. Generally, wave models such as SWAN (Wood et al. 2000), BOUSSINESQ models (Bayram & Larson 2000) and Volume of Fluid (VOF) models (Wu et al. 1994) are used for this purpose. The difficulty, however, consists in assessing the associated uncertainties which are required for the implementation of any PRA-based design of coastal flood defences. It should also be kept in mind that large uncertainties already occur in assessing the waves in deep water.

Assuming a normal distribution and defining the uncertainty of a variable x by the coefficient of variation $\sigma'_x = \sigma_x/\overline{x}$ (σ_x = standard deviation and $\overline{x}$ = mean value), very approximate orders of magnitude of the uncertainties of incident wave parameters derived from wave hindcasting and calibrated by field measurements is given in Table 1where H is the wave height, T the wave period and θ the incident wave angle (Kamphuis 1999).

Table 1. Uncertainties of wave parameters (Kamphuis 1999)

Coeff. of Variation σ'_x	σ'_H	σ'_T	σ'_θ
Deepwater waves	0.3	0.3	0.9
Shallow water waves	0.45	0.3	1.0

For further details refer to Goda (1994a) which probably represents the most detailed reference yet available on uncertainties of design wave heights. In fact, the various sources of uncertainties have been systematically identified. For some classes of uncertainties, orders of magnitudes and even formulae are proposed to assess the coefficient of variation. Nevertheless, much remains to be done in this respect.

A further important issue is the threshold at which research to improve the accuracy of design waves should be stopped. In fact, the uncertainties of wave heights may diminish the benefits of research effort in improving the accuracy beyond a certain threshold. Goda (1994b) suggested for instance a limit corresponding to a coefficient of variation of about 5 % as a reasonable value. In this respect, the concept proposed by Goda (1994b) to judge the order of magnitude of accuracy and the research efficiency with regards to the final accuracy is highly recommended as a departure basis.

For the integration of all the data related to topographical, hydraulic, structural and socio-economic boundary conditions, an appropriate Geographic Information System (GIS) can be used which should also include indications on uncertainties. The GIS-maps and data include topography and morphology, waves and water levels, defence structures and defence schemes, land use and distribution of population and assets, historical damages such as flood penetration depths and their consequences etc.

Analysis of failure modes, breach initiation and flood wave propagation

Once the topographic, hydraulic, structural and socio-economic boundary conditions have been determined, the next step consists in the systematic identification and analysis of all relevant failure modes likely to lead to flooding, including the associated hydraulic loading.

In the case of a dike for instance, flooding may be induced as a result of a dike breaching which can be initiated

1. from the seaward side through repeated wave impacts progressively eroding the structure, through wave uplift displacing revetment elements and through shear stresses induced by run up/down velocity (Figure 9)
2. from the landward side through infiltration, overflow, wave overtopping or a combination of both which may lead to piping, sliding of the rear slope revetment and sliding failure (Figure 9).

Most of the dike breaches which occurred during the catastrophic surges of 1953 in the Netherlands and of 1962 in Germany were initiated from the landward side - essentially by wave overtopping. (Oumeraci & Schüttrumpf 1999). Therefore and because of the limited extent of the paper, only the problems associated with wave overtopping and breach initiation from the landward side will be briefly discussed thereafter, before addressing the problems related to breach growth, flood wave propagation and subsequent damages in the next section.

Wave overtopping

In the past, wave overtopping of coastal defence structures has been addressed in terms of predicted time averaged overtopping rate as compared to some tolerable overtopping rates for functional and structural safety. However, time averaged overtopping quantities rarely represent suitable parameters to describe structural or functional safety. Therefore, the available admissible average overtopping rates must be questioned.

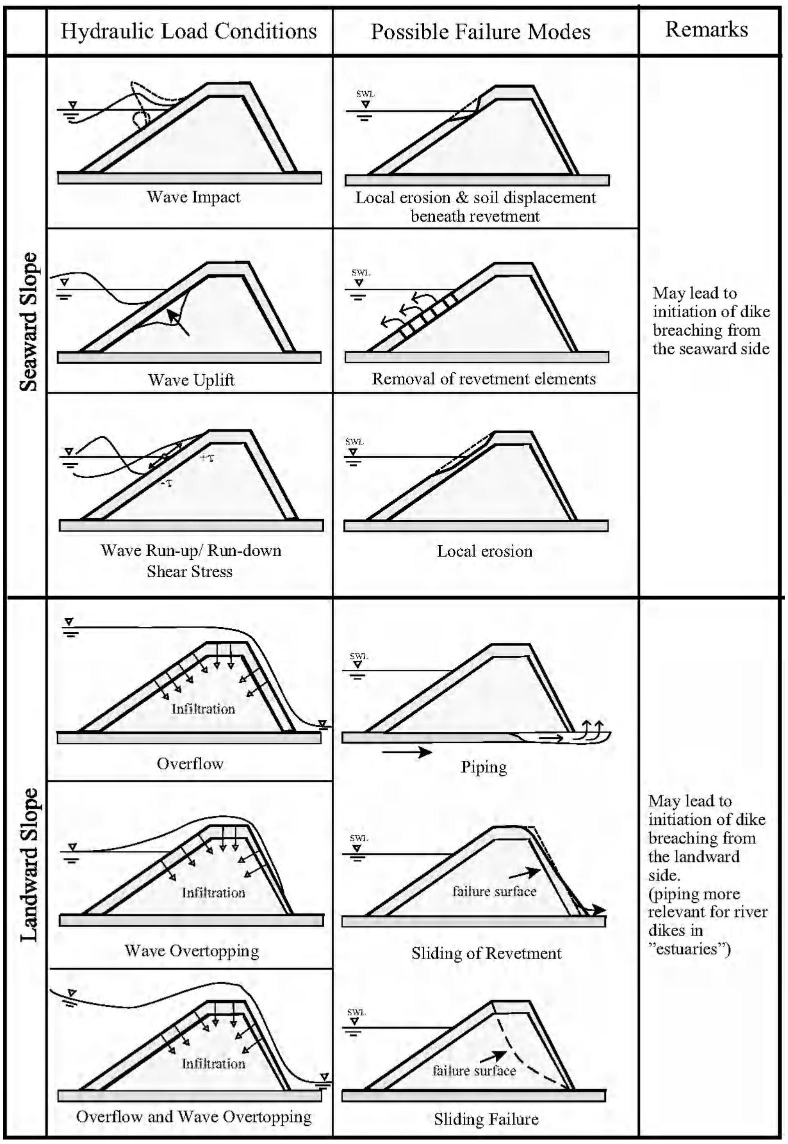

Figure 9. Possible failure modes initiating dike breach from seaward and landward side

Very recently, some attempts have been made to address overtopping in terms of individual overtopping volume (per wave) by proposing a relationship between individual and average overtopping quantities based on the assumptions of a Rayleigh distribution of the number of overtopping waves and a Weibull distribution of the individual overtopping volumes (Franco & Franco 1999). Although such relationships, which also account for the type and shape of the defence structure, are valuable to translate the traditional average quantities into individual maximum overtopping rates, future research should rather be directed towards the full description of the flow field associated with wave overtopping. A first attempt in this direction has been made by Schüttrumpf & Oumeraci (1999) who have been performing an extensive

small- and large-scale study to describe the detailed flow field associated with the overtopping of sea dikes. This also includes numerical modelling using the VOF-concept.
In fact, the knowledge of the detailed flow field associated with wave overtopping will enable to derive any type of loading (pressure, flow velocity and shear stress at any location) relevant for breach initiation.
A further important research issue is the effect of shallow foreshore on wave overtopping. Very often the natural wave spectra in such shallow foreshores are double or multi-peaked, so that the question arises on which characteristic wave heights and wave periods of the multi-peaked-spectra are most suitable to describe wave overtopping. Results of ongoing experimental investigations (Oumeraci et al. 2000b, Oumeraci 2000) have shown, that the wave period $T_{m-1,0}$ is more relevant than the peak period T_p of the entire spectrum. A more systematic examination of the influence of the lower frequency components (e.g. surf beat) on wave overtopping is also needed.

Breach growth and flood wave propagation

When simulating flood wave propagation and its devastating effects in the protected area, one of the major uncertainties arises from assessing the initial conditions of the flood wave which are essentially governed by the development of the dike breach.
The large experience available in dam engineering with dam-break flood wave models cannot be simply extrapolated to coastal flood defences, due to several reasons such as (i) the initial conditions of the flood wave which interacts with the breach growth, (ii) the limited breach width along the defence line and (iii) the 3D-character of the flood wave in a coastal plain. Therefore, substantially new knowledge towards the physical understanding and proper modelling of the breaching process must be generated before embarking into the numerical modelling of flood wave propagation and its effects on typical obstacles in the protected areas.
Due to the very strong interaction between the expected extreme hydrodynamic conditions (high water levels, strong currents and high storm waves) and soil strength parameters (large Shield's parameter, variable shear strength etc.) associated with very high erosion and transport rate during the breaching process, serious scale effects would be expected, if common small-scale models are used. On the other hand, it will not be possible to achieve the required understanding of the physical processes by using only field experiments for which the control of the forcing functions (water levels, currents and waves) and the boundary conditions cannot be controlled (also too expensive and too time consuming!). Therefore, hydraulic model tests at almost full-scale in a large wave facility will remain the sole alternative.
Since the growth of a breach initiated from the seaward side and that initiated from the landward side may differ, both cases must be experimentally examined (Figure 10). Based on the experimental results, numerical models to simulate both cases must be developed which are essential to obtain the initial conditions for the simulation of the flood wave propagation in the protected area. Once these initial conditions are properly determined, suitable numerical models (e.g. TELEMAC) exist which can be used for the simulation of the flood wave propagation. However, further research is also needed to incorporate in these models the destructive effects of the flood wave propagating in the protected area.

Integration methodology for flood risk prediction

The existing methods for the evaluation of the most relevant failure probabilities of individual components of a flood defence system must be further developed.
Much more work remains to be done with respect to the flooding probability due to the failure of the entire defence systems. The same applies for the assessment of the expected damages in the protected area. Therefore, the general methodology schematically illustrated by Figure 11 is proposed for this purpose. It integrates all the data and information resulting from the analysis of failures and their interactions, as well as from the subsequent flood wave propagation and its damaging effects in the protected area. Figure 11 shows that both cross sectional and plan view consideration of the flood defences and the protected area are indispensable.

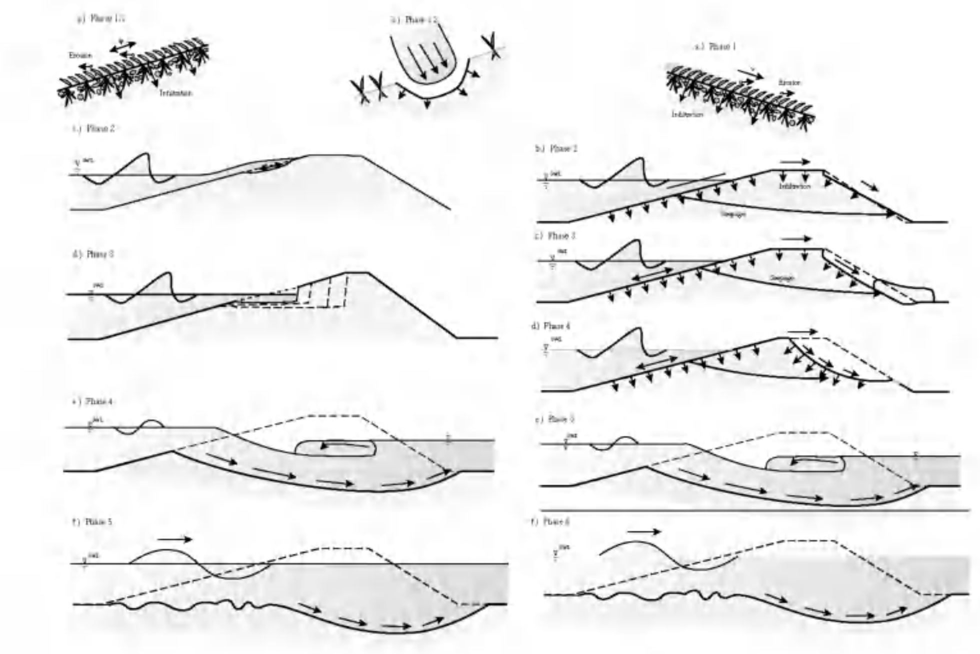

Figure 10. Stages of dike breaching (Oumeraci and Schüttrumpf 1999)

A: Breach initiated from seaward side **B: Breach initiated from landward side**

The methodology requires the use of component reliability models as well as models for the reliability of the entire flood defence scheme which consists of components with given material, cross sections and lengths. Links between the flood defence scheme components and between the protected areas with various vulnerability levels must be taken into account.

The effect of spatial correlation to account for the effect of influencing factors such as the longshore segmentation of the defence components is also important. The segmentation of the defence may become a crucial step. The degree of spatial correlation between components will depend upon the respective distance along and across shore between the defence components and on how they are tied to each other in plan view (links, bonds, etc.). Therefore, due consideration of both cross sectional representation and along shore representation of components are necessary to formulate an appropriate correlation function. As an overall result of the first step shown in Figure 4, the flood risk associated with the area protected by a given flood defence scheme is obtained (Figure 11) step, i.e. the evaluation of the acceptable flood risk, is addressed in the following section.

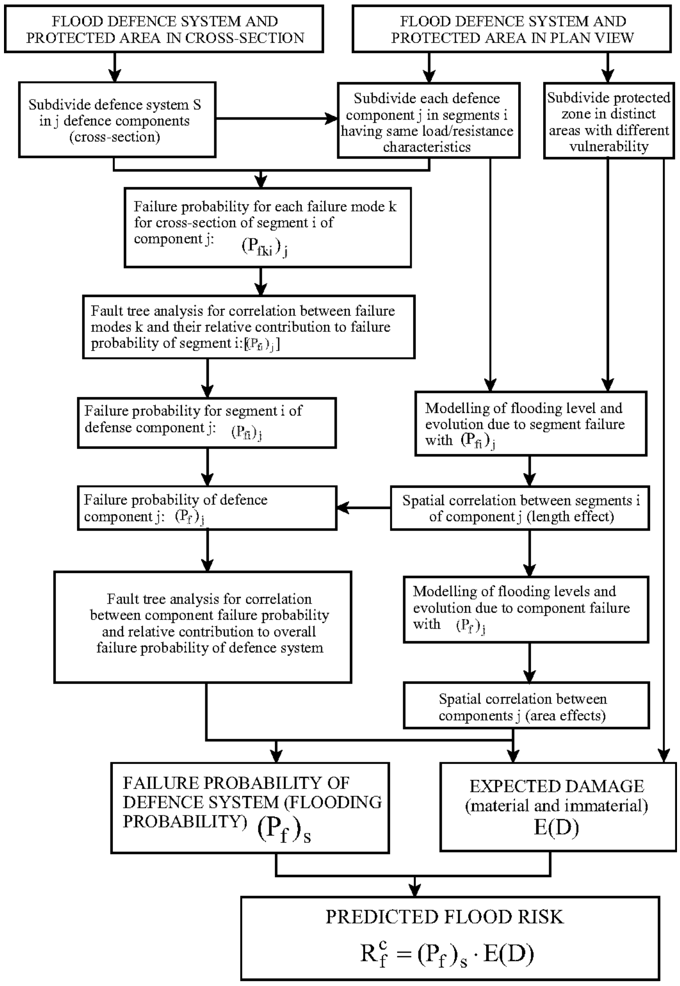

Figure 11. Integration methodology for the prediction of flood risk

Evaluation of acceptable flood risk

General methodology and framework for acceptable flood risks

Since the ALARP principle (**A**s **L**ow **A**s **R**easonably **P**racticable) is a widely accepted concept across most disciplines for the evaluation of acceptable risk, it is also recommended for the design and safety assessment of flood defence systems. However, further developments and extensions are necessary to overcome the disadvantages of the conventional ALARP approach. Candidate issues for such extensions and further developments are for example:

1. *introduction of uncertainty:* this is in fact very important as a high uncertainty of the risk may be caused by a high uncertainty of the probability of the event under consideration of/and by a high uncertainty in the consequences of that event. A high uncertainty in a very low risk is more acceptable than a comparably lower uncertainty in a very high risk (Figure 12)
2. *introduction of weight factors:* this is important to account for differences in the acceptance/penalisation of certain risks as compared to others and to achieve a better consensus on the acceptable risk across many disciplines (car traffic risk more accepted than the risk with the same value

for a dike breach and 1000 hazard events with 1 fatality/event are more accepted than 1 hazard event v

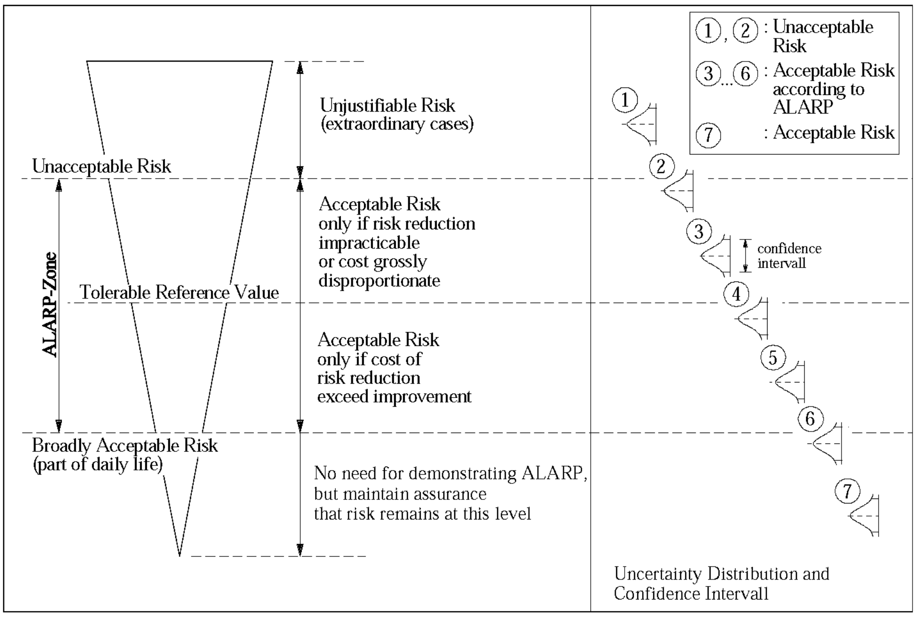

Figure 12. Introduction of uncertainty into the ALARP concept

Evaluation of tangible and intangible losses

In order to achieve a wide consensus on the acceptable flood risk in accordance with acceptable risks in other disciplines (e.g. dam engineering, offshore engineering, transportation, nuclear power plants), it is indispensable that the various methods, rules and tools to be developed in the advanced ALARP framework are robust and transparent. To increase this transparency and to enable a better comparison with the acceptable risks in other disciplines, the acceptable (target) flood risk R_f^t is defined as a product of the acceptable (target) flooding probability P_f^t and the acceptable (target) damages or losses A(D) (Figure 3). If the damages are expressed in monetary terms the target flooding probability P_f^t may be formulated as a cost optimisation problem (Figure 13). In addition, however, the uncertainties resulting from the assumptions and cost calculations must explicitly be taken into account within the overall probabilities framework.

Most of the difficulties arise when trying to evaluate the so-called intangible losses such as human injury, loss of life, environmental and cultural losses caused by flooding.

Although the valuation of human life is questionable from the ethical view point, the problem is often formulated in terms of the amount society is willing to pay for saving life. Values between 1 to 10 million US$, depending on considerations associated with aversion of risk, have been reported. Various methods to evaluate intangible losses are available in the literature which can systematically be analysed to derive the approach most appropriate for coastal flooding.

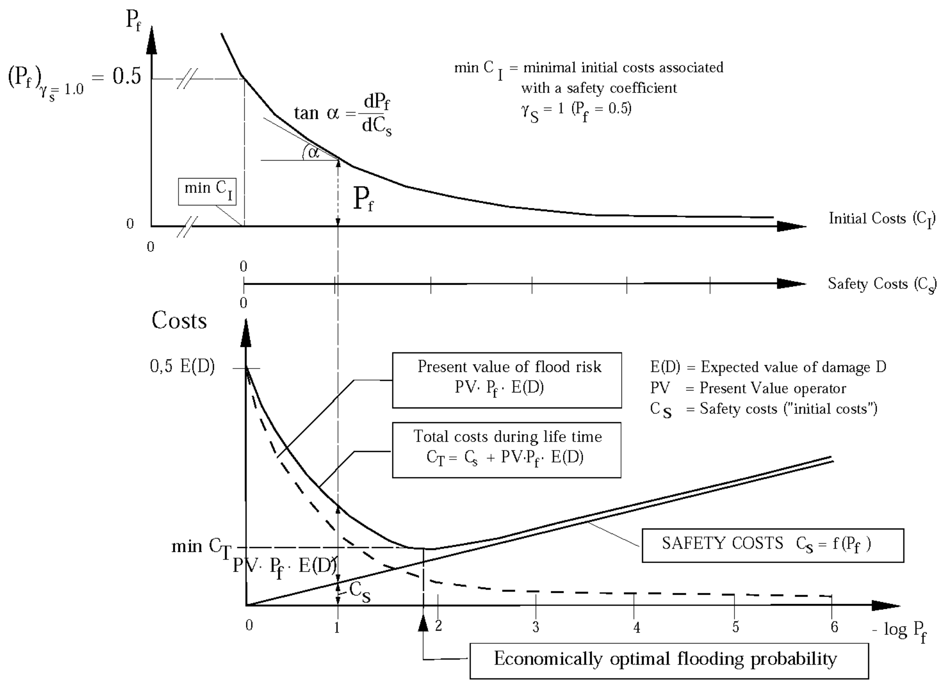

Figure 13. Formulation of terget flooding probability as a cost optimisation problem (Oumeraci et al. 200a)

Integration of method for acceptable risk evaluation

The general procedure for the evaluation of the acceptable flood risk within an advanced ALARP framework is tentatively summarised in Figure 14. It includes seven steps requiring the use of techniques and tools which exist already in Cost-Benefit-Analysis (CBA), Reliability Theory and Multi-Criteria Decision Theory or needs new/further development.
The major problems with most of these methods is that they are very complex and hardly understandable for most prospective users. The greatest challenge will therefore consist in simplifying as much as reasonably practicable, i.e. without loosing the important aspects.

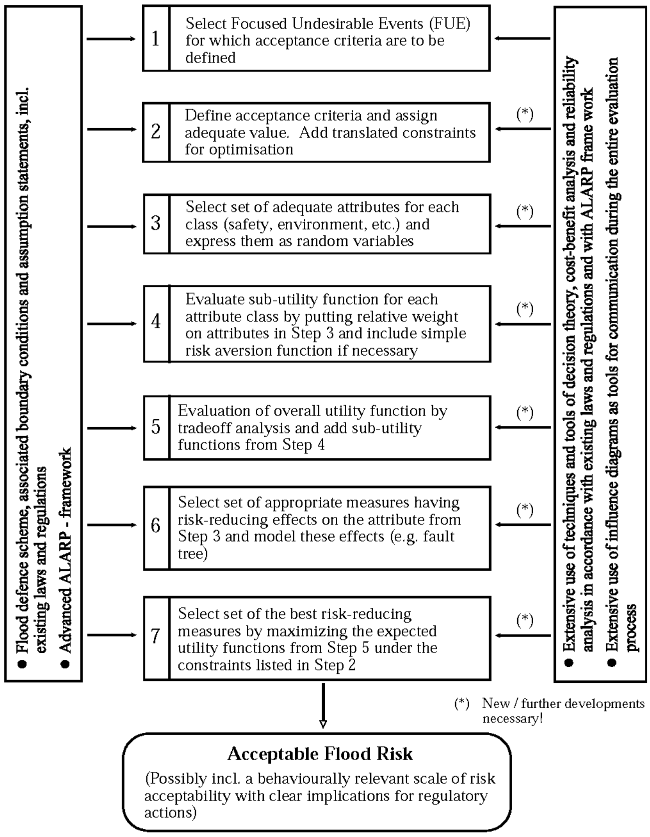

Figure 14. Flow diagram for acceptable flood risk evaluation

Risk scale, discussion and suggestions for research

Risk scale for decision making

Once the predicted flood risk (R_f^c) and the acceptable flood risk (R_f^t) are obtained, a measure of the flood risk level which is appropriate for the decision making under consideration can be formulated as a function of costs and further intangible losses. For instance, a risk scale $G = (R_f^c - R_f^t) / R_f$ is tentatively proposed in Figure 3, showing that optimum risk level is obtained for $G = 0$. Negative G-values mean overdesign while positive G-values mean underdesign. In both cases, penalty curves provide the costs or losses associated with every over- and underdesign.

Comparison of proposed PRA framework with other approaches

The advantages of the prospective PRA framework, together with the new methodologies and techniques which would result, as compared to the present design practice based on design water level and to the newly emerging pseudo-risk assessment approaches are summarised in Table 2.

Suggestions for research

In addition to the suggestions provided in the previous sections, the following questions must be answered:

1. How to simplify the developed methods (**As** **S**imple **A**s **R**easonably **P**racticable); i.e. without loosing the important aspects? This is a very important issue towards facilitating the transition from the older era - where conservatism, local tradition and local authorities prevailed - to a new era of quantitative analysis methods for design and management which are very sophisticated in their essence and background, but should made simple in their application.
2. How to conduct efficiently and cost-effectively a quantitative risk assessment including two steps: a preliminary approach under the constraints of the available (usually very limited) data to identify the focus points and to optimise the next more detailed and costly step, namely the new proposed comprehensive PRA-approach.
3. How to demonstrate the superiority of the new approach as compared with the present approaches and with the diverse newly emerging pseudo-risk assessment and management approaches (Table 2), which ignore or grossly simplify the underlying physics of the processes involved, particularly those associated with extreme situations like breaching? For this purpose indicators to measure the benefits of the new methods must be developed.
4. How to apply the new proposed PRA framework to estimate the threshold between sustainable and non-sustainable flood protection? This may particularly be made possible through the high level of integration, including the evaluation of direct and indirect costs, loss of life, environmental, cultural and further intangible losses? This aspect is expected to particularly contribute to overcome the major present barrier to sustainable design and management of flood protection which at present certainly lies in the lack of a rational, transparent, impartial and integrated framework that could be broadly accepted at multiple scales, including local, national and transnational levels.
5. How the new PRA-Approach can be used as a meaningful yardstick for determining priorities in design, management and maintenance as well as in scientific research designed to held developing coastal protection schemes meeting sustainability criteria? For this purpose, the reliability indices and the penalty functions obtained from PRA may be used, together with the sustainability thresholds suggested in the previous item 4.
6. How to make best use of the new PRA framework towards the implementation of a new transparent and unified safety concept for the design of coastal flood defences which also includes the management of the remaining risk (monitoring and inspection strategy, review and safety evaluation update strategy, maintenance and repair strategy and emerging strategy) as an integral part of the design processes?

Table 2. Comparison of new PRA framework with other approaches

Comparison Criteria	Present Practice Based on Design Water Levels	Newly Emerging Pseudo Risk Assessment and Management Approaches	Proposed New PRA Framework
Physical Background for the Prediction of Flood Hazards / Flood Risk	Ignore the joint probability of storm water levels and waves		joint probability of water level and waves represents key input
	• ignore totally or partially the failure modes likely to lead to flood hazards • unable to quantify flooding damages	• grossly simplify or ignore underlying physics associated with breach initiation and breach growth	• account for all relevant failure modes likely to lead directly or indirectly through breach initiation to flood hazards • detailed prediction of breach growth and its effect on flood wave propagation and subsequent damages represents a key innovative issue of the new PRA-framework • breach initiation is considered both from seaside and leeward side • the linkage between the failure modes in cross sectional and plan view representation is properly accounted for (Figure 11)
Social Safety, Economic and Environmental Background of Accepted Flood Risk	Only implicitly and arbitrarily considered	Explicitly considered, but lacking any clear and systematic framework and methodology	• based on a systematic and transparent framework and methodology for acceptable flood risk, with comparison to other risks • clear and systematic integration in a complimentary manner of all aspects into the evaluation of acceptable flood risk (Figure 4)
Level of Integration and Complexity	very low level of (implicit) integration and complexity	Relatively low complexity and moderate level of integration, but lacking explicit sound backgrounds	Very high level of complexity and integration through explicit involvement of direct and indirect economic losses, loss of life, environmental and further intangible losses in the evaluation of acceptable flood risk and involvement of all aspects contributing to the flooding hazards in the predicted flood risk

Comparison Criteria	Present Practice Based on Design Water Levels	Newly Emerging Pseudo Risk Assessment and Management Approaches	Proposed New PRA Framework
Credibility Level of Safety / Risk Assessment & Management	• incorrect safety assessment • questionable overall safety coefficients • not appropriate at all for risk management	• overall risk figures evaluated on non-transparent basis • inappropriate evaluation of remaining risk making an effective management almost impossible	• clear evaluation of remaining risk for which efficient managing risk reducing measures can be developed • clear contribution of each aspect and hazard to the overall risk, thus facilitating the prioritization of management measures and investments for risk reduction
Potential for Acceptability, including Synergetic Power For Sustainability	Very limited and inconsistent with sustainable protection (accepted only at local level)	might be appropriate as a first step, before embarking into the detailed new PRA-framework	• through its highly integrative nature the new framework is ideal to help evaluating sustainability thresholds • high acceptability by end users when simplification or/and transfer into easy to use software packages have been achieved
Potential for Development of Guidelines, Software Packages and Operational Management Tools	possible only at local level, but impossible at transnational level	might be adequate at feasibility level, before embarking into detailed new PRA-framework	• indispensable for the development of any rational and modern integrated design and management guidelines • possibility to simplify the application by prospective end users through the development of practical software packages and through transfer of the methodologies into tools for different design levels (feasibility, preliminary, detailed and research level) • technical basis for the development of new operative management tools (e.g. new warning system)

Concluding remarks

Although there is still a long way to go and many mountains to climb, the proposed PRA-based framework and the prospective methodologies that would result are expected to help moving sustainable design of coastal flood defences from an academic debate into the realm of concrete work, performance and return. It will also help to overcome the conservatism of isolated national/regional safety cultures which typify the past and present situation in the design of coastal flood defences. Moreover, the proposed PRA-based framework has the capability to ignite the awareness of the coastal engineering community that time is ripe for a synergetic transnational partnership to forge the transition to a more integrated systematic and

transparent design framework which is based on a physically, socio-economically and environmentally sound ground to meet the sustainability requirements.
One of the key features of the proposed framework is the focus on the underlying physics of the processes likely to lead to devastating damages (e.g. breach initiation, breach growth, flood wave propagation and its damaging effects) as well as on the explicit account of all uncertainties. This indeed makes all the difference with the newly emerging pseudo-risk assessment procedures which ignore or grossly simplify these important aspects.
Since the new proposed framework is intended to also provide a robust and transparent methodology to evaluate the acceptable risk, taking into account tangible and intangible losses associated with coastal flooding, the results will have clear implications for regulatory actions. In fact, the results will help developing unified safety concepts and thresholds between sustainable and non-sustainable flood protection schemes.

Besides further challenges associated with methodological (e.g. linkage of elicited expert opinions and calculations, linkage of failure modes across and along the defence line, integrated method for acceptable flood risk) and the modelling aspects (e.g. breaching, damaging effects of flood wave), the greatest challenge will certainly be to simplify as much as reasonably practicable (e.g. without loosing relevant aspects!), so that the methods will be comprehensible and affordable by most prospective end users.

Acknowledgements

The concept and ideas presented in this paper were developed over years during the course of PROVERBS (MAS3-CT95-0041) and other research projects supported by the European Community, the German Research Council (DFG) and the Federal Ministry for Science, Education, Research and Technology (BMBF / Germany). The author gratefully acknowledges this support as well as the contributions of my co-workers and other partners to these research projects.

References

Bayram, A. and Larson, M. (2000). *Wave transformation in the nearshore zone: comparison between a Boussinesq model and field data.* Coastal Engineering, Vol. 39, Nos 2/4, pp. 149-172.

Cooke, R.M. (1991). *Experts in uncertainty.* Oxford University Press, New York, 321 pp.

Franco, C. and Franco L. (1999). *Overtopping formulas for caisson breakwaters with non breaking 3D waves.* ASCE, J. Waterway, Port, Costal and Ocean Eng., Vol. 125, No. 2, pp. 98-108.

Goda, Y. (1994a). *On the uncertainties of wave heights as the design load for maritime structures.* Proc. Intern. Workshop on wave barriers in deep waters. Yokusaka, Port and Harbour Research Institute (PHRI), pp. 1-18.

Goda, Y. (1994b). *A plea for engineering-minded research efforts in harbour and coastal engineering.* Proc. Intern. Conf. Hydroport ´94, Yokosuka/Japan, pp. 1-21.

Kamphuis, J.W. (1999). *Marketing Uncertainty.* Proc. COPEDEC V., Cap Town, pp. 2088-99.

Oumeraci, H. and Muttray, M. (1999*). Design wave parameters in front of structures with different reflection properties.* LWI-Research Report, TU Braunschweig, 75 pp. (in German).

Oumeraci, H.; Kortenhaus, A.; Allsop, N.W.H.; De Groot, M.B.; Crouch, R.S.; Vrijling, J.K. and Voortman, H.G. (2000a). *Probabilistic Design Tools for Vertical Breakwaters.* Forthcoming Book, Balkema, pp. 300 (in prep.).

Oumeraci, H.; Schüttrumpf, H.; Sauer, W.; Möller, J. and Droste, T. (2000b). *Physical model tests on wave overtopping with natural sea states. 2D model tests with single, double and multi-peaked natural wave energy spectra*. LWI-Research-Report No. 852, TU Braunschweig.

Oumeraci, H. (2000). *The Sustainability Challenge in Coastal Engineering*. Keynote Lecture in Proc. Intern. Conf. Hydrodynamics ICHD ´2000, Yokohama, Japan, 27 pp.

Oumeraci, H. and Schüttrumpf, H. (1999). *Review Analysis of failures of sea dikes* (in German). LWI Research Report, 53 pp.

Owen, M.; Hawkes, P.; Tawn, J. & Bortot, P. (1997). *The Joint Probability of waves and water levels: a rigorous but practical new approach*. Proc. MAFF Workshop, Keele, U.K. pp. B4.1 - B4.10.

Schüttrumpf, H. & Oumeraci, H. (1999). *Wave overtopping at sea dikes*. Proc. HYDRALAB Workshop, Hannover, Pub. Forschungszentrum Küste (FZK), pp. 327-334.

Wood, D.J.; Muttray, M. & Oumeraci, H. (2000). *The SWAN model used to study wave evolution in a flume*. Ocean Engineering (in Press).

Wu, N.T.; Oumeraci, H. and Partenscky, H.-W. (1994). *Numerical modelling of breaking wave impacts on a vertical wall using the volume-of-Fluid Method*. ASCE, Proc. 24th Intern. Conf. Coastal Eng., Kobe, Japan.

Probabilistic Design and Maintenance of Water Defense Systems

J.K. Vrijling
Delft University of Technology, Delft Cluster, Faculty of Civil Engineering and Geosciences, Hydraulic Engineering Section.
e-mail: J.K.Vrijling@ct.tudelft.nl

Abstract

After the disaster in 1953 a statistical approach to the storm surge levels was chosen and an extrapolated storm surge level would be the basis for dike design. In recent decades, the development of reliability theory made it possible to assess the flooding risks taking into account the multiple failure mechanisms of a dike section and the length effect. It is pointed out that economic activity in the protected areas has grown considerably since the 1950s and that even more ambitious private and public investments, particularly in infrastructure, are planned. Moreover the safety of a growing population is at stake. These considerations justify a fundamental reassessment of the acceptability of the flood risks.

Introduction

Approximately half of the Netherlands lies below sea level and is protected against flooding by primary dikes and other water-retaining structures. Extensive systems of drainage channels and storage basins with associated water-retaining structures and pumping capacity ensure artificial management of surface and groundwater levels. In the Middle Ages the dikes were designed at the highest known storm surge plus one meter additional freeboard. This practice had to be abandoned in the design of the Afsluitdijk, where no observations were available. The famous Prof. Lorentz predicted the increases in tidal level and to these observations of wind setup at the Frisian coast were added. After the disaster in 1953 a statistical approach to the storm surge levels was chosen and an extrapolated storm surge level would be the basis for dike design. It will be described how the Deltacommittee optimised these safety levels, that were expressed in terms of the return period of high water levels which must be withstood by the primary dike system.

Since 1980 the development and the application of reliability theory made it possible to assess the flooding risks taking into account the multiple failure mechanisms of a structure. Dutch hydraulic engineers were among the first to apply this theory in the practical design of structures. Reliability models were first used during the design and the construction of storm surge barrier in the Eastern Scheldt in 1976 and later in the design of storm surge barrier in the Nieuwe Waterweg.

In approximately 1979 a project was started to apply the probabilistic methods to the design of dikes in general. The development of a complete approach to water defense systems took a considerable time. Recently the approach was tested on four polders or dike rings. The probability of flooding of these polders was calculated and insight in the weak spots was gained. Some results will be presented below. It is expected that the results of the calculations will stimulate the political debate if the present safety level of the water defense system is still sufficient. This question should be posed because the economic activity in the protected areas has grown considerably since the 1960s. Moreover ambitious private and public investments, particularly in infrastructure, are planned. The national economy has and will become more vulnerable to flooding. In addition the safety of a growing number of inhabitants is at stake. In future, the environmental consequences of flooding and the potential effects on nature and cultural heritage will play a larger role in assessing the required scale of flood protection. The image of The Netherlands as a safe country to live, work, and invest is an important factor to consider. This may justify a fundamental reassessment of the acceptability of the flood risks and the development of a plan to improve the effectiveness of the water defense system in the course of the coming decades.

The approach to dike design since the Delta Committee

The present approach to the safe design of dikes is based on the findings of the Delta Comittee, that were published in 1960. In 1953 large parts of the South-western Delta of the Netherlands were flooded by an extreme storm surge. Apart from considerable economic losses 1800 people lost their lives. The main cause of the failure during the storm of the 1st of February of 1953 was overtopping followed by the erosion and sliding of the inner slope of the dikes. Besides the development of the Delta-plan to shorten the coastline by closing the estuaries, the Delta Committee focussed on the design of dikes. Two main improvement were proposed: 1. increase the design water level and consequently the crest height of the dike 2. flatten the inner slope of the dike to 1:3. The new design water level was established in two steps. First the observations of the previous hundred years were statistically analysed and extrapolated to levels never exceeded before. Secondly an economic optimisation of the design water level was performed. On the one hand the damage that a flooding of Central-Holland would cause was estimated, on the other the cost of increasing the height of the dikes was calculated. Because a higher and more expensive dike leads to a reduction of the probability of flooding and therewith of the occurence of damage an optimal design water level could be found (Figure 1):

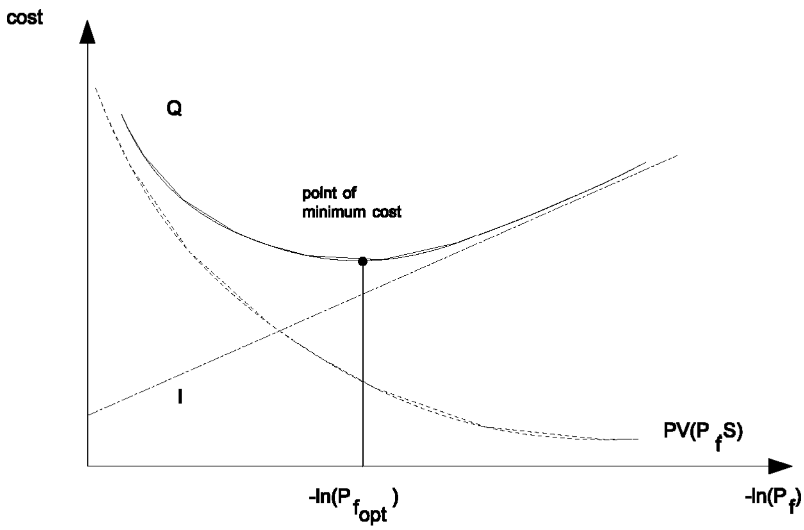

Figure 1. The economically optimal probability of failure of a structure

The design water level for Central-Holland was fixed at NAP + 5m with a return period of 10,000 years. Because the flood damage in more rural areas than Central-Holland would be less, shorter return periods of 4000 and 3000 years were chosen for these polders. Today the Netherlands are divided in 53 polders each with a specified design return period or frequency. The Delta Committee knew that other failure mechanisms than overtopping and sliding could be dangerous, therefore additional design requirements were formulated in a classical way.

The probabilistic approach of flooding

Since 1980 the awareness grew that the probability of exceedance of the design water level, the design frequency or the reciprocal of the return period is not a good predictor of the probability of flooding. Normally the dike crest exceeds the design water level by some measure, thus the probability of overtopping is smaller than the design frequency. But some parts of the dike may already be critically loaded before the design water level is reached. Waterlogging may lead to slide planes through the dike or piping may undermine the body of the dike, with sudden failure as a consequence. Another danger is that sluices and gates are not closed in time before the high water. In short there are more failure mechanisms that can lead to flooding of the polder than overtopping (Figure 2). Moreover the length of the dike ring has a considerable influence. A chain is as strong as the weakest link. So a single weak spot determines the actual safety of the dike ring.

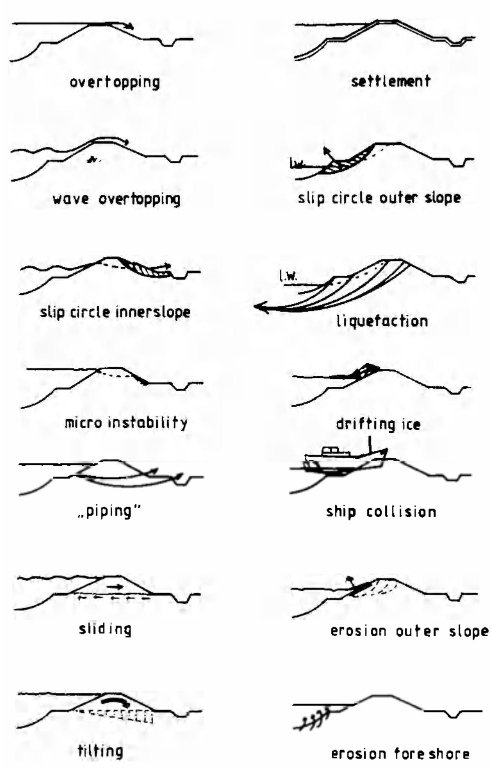

Figure 2. Failure modes of a dike

The probabilistic approach aims to determine the probability of flooding of a polder and to judge its acceptability in view of the consequences. As a start the entire water defence system of the polder is studied. Typically this system contains sea dikes, dunes, river levees, sluices, pumping stations, high hills, etc (Figure 3).

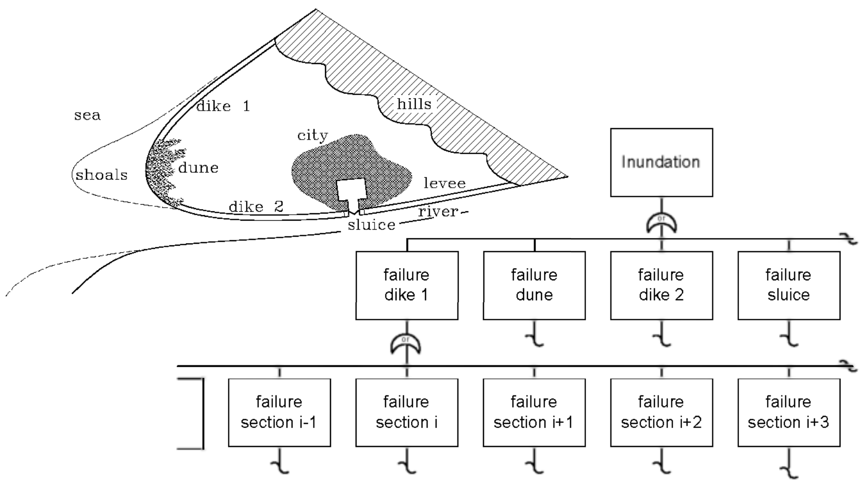

Figure 3. Flood defence system and its elements presented in a fault tree

In principle the failure and breach of any of these elements leads to flooding of the polder. The probability of flooding results thus from the probabilities of failure of all these elements. Within a longer element e.g. a dike of 2 km length, several independent sections can be discerned. Each section may fail due to various failure mechanisms like overtopping, sliding, piping, erosion of the protected outer slope, ship collision, bursting pipeline, etc. The relation between the failure mechanisms in a section and the unwanted consequence flooding can be depicted with a faulttree as shown in Figure 4.

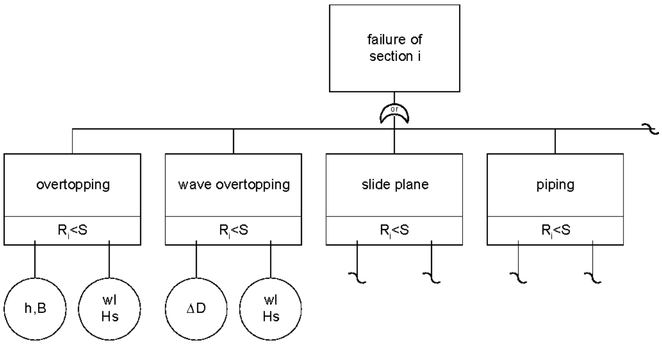

Figure 4. A dike section as a series system of failure modes

The failure probabilities of the mechanisms are calculated using the methods of the modern reliability theory like Level III Monte Carlo or Level II advanced first order second moment calculations. The possibility to treat the human failure to close for instance a sluice in conjunction with structural failure is seen as a considerable advantage of the probabilistic approach (Figure 5).

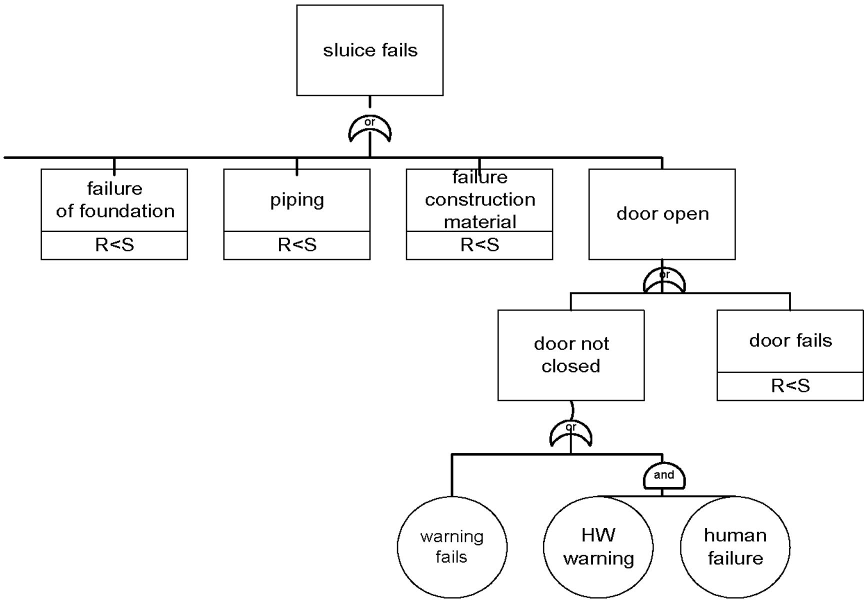

Figure 5. The sluice as a series system of failure modes

In the reliability calculations all uncertainties are taken into account. Three classes are discerned. The intrinsic uncertainty is characteristic for natural phenomena. Storm surge levels and extreme river discharges are intrinsically uncertain. Also the uncertainty of the properties of the sub-soil and of construction materials falls in this class. Model uncertainty describes the imperfection of the engineering models in predicting the behaviour of river courses, dikes and structures. The comparison of predictions and observations provides an estimate of this uncertainty. If such data lack, engineering judgement is the only feasible replacement. Statistical uncertainty is caused by the lack of data. These data are used to estimate the parameters of the probability distributions depicting the intrinsic uncertainty. Due to the paucity of data the estimated values of the parameters are statistically uncertain. Because all uncertainties are included in the calculations of the failure probability the latter is not singly a property of the physical reality but also of the human knowledge of the system.

The consequence is that the safety of the dike system as expressed by the calculated probability of flooding can be improved by strengthening the weakest dike but also by increasing our knowledge. The result the calculated probability of flooding of the polder is presented in Table 1..

Table 1. Calculation table for the overall probability of flooding

section	overtopping	piping	etc.	total
dike section 1.1	$p_{1.1}$(overtopp.)	$p_{1.1}$(piping)	$p_{1.1}$(etc.)	$p_{1.1}$(all)
dike section 1.2	$p_{1.2}$(overtopp.)	$p_{1.2}$(piping)	$p_{1.2}$(etc.)	$p_{1.2}$(all)
etc.	..	..	..	..
dune	p_{dune}(overtop.)	p_{dune}(piping)	p_{dune}(etc.)	p_{dune}(all)
sluice	p_{sluice}(overtop.)	p_{sluice}(piping)	p_{sluice}(etc.)	p_{sluice}(all)
total	p_{all}(overtop.)	p_{all}(piping)	p_{all}(etc.)	p_{all}(all)

The last column of the table shows immediately which element or section has the largest contribution to the probability of flooding of the polder under study. Inspection of the related row reveals which mechanism will most likely be the cause. Thus a sequence of measures can be defined which at first will quickly improve the probability of flooding but later runs into dimishing returns.
As stated earlier the approach is tested on four polders of which the most important is shown here as an example. For Central Holland the 48 weakest dike sections, 4 dune sections and 6 structures were selected. The analysis showed that the probability of human failure to close a sluice structure at Katwijk dominated the system with a probability of 1/600 year. If it is assumed that this problem is solved in some way, then the probability of flooding becomes 1/2000 year. Piping under a dike near the village of Moordrecht determines this. If this piping problem can be solved, the probability of flooding sinks to 1/30,000 year. Now the dune along the North Sea near the village of Monster forms the weakest link. Improving the strength of this dune seems to reduce the flooding probability to 1/75,000 year. Although the example shows the results and the approach clearly, the last result is questionable as it hinges on the selection of the 48+4+6 elements at the start. Weaker links may yet be detected at this level.

The consequences of flooding

One of the tasks of human civilisations is to protect individual members and groups against natural and man-made hazards to a certain extent. The extent of the protection was in historic cases mostly decided after the occurrence of the hazard had shown the consequences. The modern probabilistic approach aims to give protection when the risks are felt to be high. Risk is defined as the probability of a disaster i.e. a flood related to the consequences. As long as the modern approach is not firmly embedded in society, the idea of acceptable risk may, just as in the old days, be quite suddenly influenced by a single spectacular accident or incident like the non-calamitous threats of the Dutch river floods of 1993 and 1995.

The estimation of the consequences of a flood constitutes a central element in the modern approach. Most probably society will look to the **total** damage caused by the occurrence of a flood. This comprises a number of casualties, material and economic damage as well as the loss of or harm to immaterial values like works of art and amenity. Even the loss of trust in the water defense system is a serious, but difficult to gauge effect. However for practical reasons the notion of risk in a societal context is often reduced to the total number of casualties using a definition as: "the relation between frequency and the number of people suffering from a specified level of harm in a given population from the realisation of specified hazards". If the specified level of harm is limited to loss of life, the societal risk may be modelled by the frequency of exceedance curve of the number of deaths, also called the FN-curve. The consequence part of a risk can also be limited to the material damage expressed in monetary terms as the Delta Committee did. It should be noted however, that the reduction of the consequences of an accident to the number of casualties or the economic damage may not adequately model the public's perception of the potential loss. The schematisation clarifies the reasoning at the cost of accuracy.

The consequences in case of the flooding of polders will be estimated from two points of view. First an estimate of the probability to die for an individual residing at some place in the polder will be given. The most practical form of presentation might be a contour plot of the risk as a function of the place in the polder. Secondly the total number of people that will drown in a flood must be estimated. Because a polder can flood due to breaches at various places and according to different scenarios a FN-curve may be the best way to present this type of risk. Thirdly the total material damage that will be caused by a flood must be estimated. For reasons mentioned above a FN-curve may also be the best way to present this type of risk. It should be noted that the Delta Committee limited itself to the expected value of the material damage. The loss of life and the material damage will be estimated using cause-effect functions based on the experiences of the 1953 disaster. This could be criticised as considerable technical progress has been made providing people with better means but making society more vulnerable. Increased as well as reduced effects can be deduced therefore the cause-effect functions have stayed unaltered for the moment. As an example the FN-curve of the Brielse polder is shown in Figure 6.

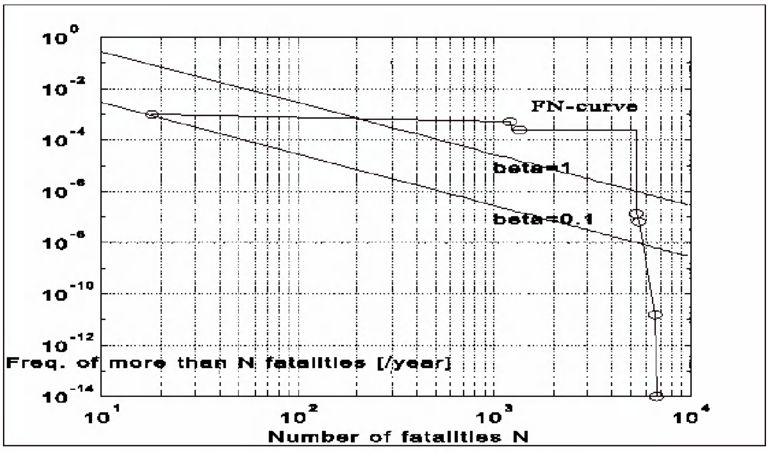

Figure 6. FN-curve for flooding of the Brielse polder

The decision problem or the notion of acceptable risk

When the probabilities of flooding of the polders of the Netherlands are calculated the question will be: Are they safe enough? The answer for this question can only come from a broad judgement of the cost of improving the defences against the reduction of the probability of flooding related to the scale of the damage. This judgement is political in essence, because the costs as well as the benefits contain many aspects that may be valued quite differently by different parties. Dike improvement costs money, cultural heritage and amenity. The valuation problem of the consequences was already discussed. However the general picture is clear. If dike improvement is very expensive a higher probability of flooding will be accepted. On the other hand if the consequence of flooding is very substantial one will aim for a smaller probability. As shown already in the example of the previous pragraph an increasing effort will lead to an improvement of the safety of the dike that diminishes in the end. Efforts made to increase the knowledge as well as efforts to make structural improvements to the dike itself can be effective. Inspection of the existing dike system improves the knowledge and reduces the uncertainty. Checking the actual state of structural elements like sluices, pumping stations, etc may be particularly effective. Further research may reduce the model uncertainty with respect to certain failure modes and some less well known parameters. In the third place comes the classical improvement by increasing the height or the width of dikes. In principle this involves much larger sums of money. Very effectively starting with the weakest sections one will quickly meet diminishing returns. Progressing with increasing cost means finally the upgrading of the dike ring to a higher standard. The reduction of the loading in the sense of water levels or wave heights is sometimes a fourth possibility. Widening the flood plain of a river to reduce the water level is a modern example. The reduction of the future consequences of flooding by adequate spatial planning or disaster management is the last and completely new avenue opened by the new approach. In order to make a choice the costs of the measures depicted above must be weighed against the reduction of the probability of flooding and the related consequences. It should be noted that the economic activity in the protected areas has grown considerably since the 1960s when the Delta Committee wrote its advice. Even more ambitious private and public investments, particularly in infrastructure, are planned. The national economy has therefore a far greater and still growing value exposed to flooding. Moreover the safety of a growing number of inhabitants is at stake. Contrary to the old days, many people are presently housed in suburbs located in the deepest areas of the polders. In future, the environmental consequences of flooding and the potential effects on nature and cultural heritage will play an increasing role in assessing the required scale of flood protection. The image of The Netherlands as a safe country to live, work, and invest is finally at stake. This justifies a fundamental reassessment of the acceptability of the flood risks in view of the costs of improvement.

The smallest component of the social acceptance of risk is the assessment by the individual. Attempts to model this are not feasible, therefore it is proposed to look to the preferences revealed in the accident statistics. The fact, that the actual personal risk levels connected to various activities show statistical stability over the years and are approximately equal for the Western countries, indicates a consistent pattern of preferences. The probability of losing one's life in normal daily activities such as driving a car or working in a factory appears to be one or two orders of magnitude lower than the overall probability of dying. Only a purely voluntary activity such as mountaineering entails a higher risk (Figure 7).

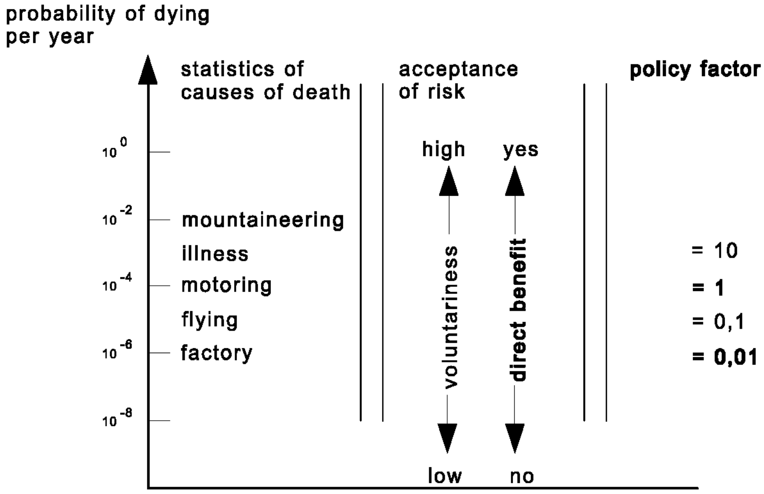

Figure 7. Personal risks in Western countries, deduced from the statistics of causes of death and the number of participants per activity

Apart from a slightly downward trend due to technical progress, of the death risks presented, it seems permissible to use them as a basis for decisions with regard to the personally acceptable probability of failure in the following way:

$$P_{fi} = \frac{\beta_i \bullet 10^{-4}}{P_{d|fi}}$$

where $P_{d|fi}$ denotes the probability of being killed in the event of an accident. In this expression the policy factor β_i varies with the degree of voluntariness with which an activity *i* is undertaken and with the benefit perceived. It ranges from 100 in the case of complete freedom of choice like mountaineering, to 0.01 in case of an imposed risk without any perceived direct benefit. This last case includes the individual risk criterion used for the siting of a hazardous installation near a housing area without any direct benefit to the inhabitants. A proposal for the choice of the value of the policy factor β_i as a function of voluntariness and benefit is given in table 2.

Table 2. The value of the policy factor β_i as a function of voluntariness and benefit.

β_i	voluntariness	direct benefit	example
100	voluntary	direct benefit	mountaineering
10	voluntary	direct benefit	motorbiking
1.0	neutral	direct benefit	car driving
0.1	involuntary	some benefit	factory
0.01	involuntary	no benefit	LPG-station

For the safety of dikes a β_i-value of 1.0 to 0.1 is thought to be applicable. The judgement of societal risk due to a certain activity should be made on a national level. The risk on a national level is the aggregate of the risks of local installations or activities. Starting with a risk criterion on a national level one should evaluate the acceptable local risk level, in view of the actual number of installations, the cost/benefit aspects of the activity and the general progress in safety, in an iterative process with say a fifty year cycle.

The determination of the socially acceptable level of risk assumes also that the accident statistics reflect the result of a social process of risk appraisal and that a standard can be derived from them. The formula should account for risk aversion in a society. Relatively frequent small accidents are more easily accepted than one single rare accident with large consequences like a flood, although the expected number of casualties is equal for both cases. The standard deviation of the number of casualties reflects this difference.

Risk aversion can be represented mathematically by adding the desired multiple k of the standard deviation to the mathematical expectation of the total number of deaths, $E(N_{di})$ before the situation is tested against the norm of $\beta_i \cdot 100$ casualties for the Netherlands:

$$E(N_{di}) + k \bullet \sigma(N_{di}) < \beta_i \bullet 100$$

where: k = 3 risk aversion index

To determine the mathematical expectation and the standard deviation of the total number of deaths occurring annually in the context of activity i, it is necessary to take into account the number of independent places N_{Ai} where the activity under consideration is carried out.

The translation of the nationally acceptable level of risk to a risk criterion for one single installation or polder where an activity takes place depends on the distribution type of the number of casualties for accidents of the activity under consideration. In order to relate the new local risk criterion to the FN-curve, the following type is preferred:

$$1 - F_{N_{dij}}(x) < \frac{C_i}{x^2} \quad \text{for all } x \geq 10$$

If the expected value of the number of deaths is much smaller than its standard deviation, which is often true for the rare calamities studied here, the value of C_i reduces to:

$$C_i = \left[\frac{\beta_i \bullet 100}{k \bullet \sqrt{N_{A_i}}} \right]^2$$

The problem of the acceptable level of risk can be also formulated as an economic decision problem as explained earlier. The expenditure I for a safer system is equated with the gain made by the decreasing present value of the risk (Figure 1). The optimal level of safety indicated by P_f corresponds to the point of minimal cost.

$$\min(Q) = \min(I(P_f) + PV(P_f \bullet S))$$

where : Q = total cost
PV = present value operator
S = total damage in case of failure

If despite ethical objections, the value of a human life is rated at *s*, the amount of damage is increased to:

$$P_{d|fi} \bullet N_{pi} \bullet s + S$$

where N_{pi} = number of inhabitants in polder *i*.

This extension makes the optimal failure probability a decreasing function of the expected number of deaths. The valuation of human life is chosen as the present value of the nett national product per inhabitant. The advantage of taking the possible loss of lives into account in economic terms is that the safety measures are affordable in the context of the national income.

In assessing the required safety of a dike system the three approaches described above should all be investigated and presented. The most stringent of the three criteria should be adopted as a basis for the "technical" advice to the political decision process. However all information of the risk assessment should be available in the political process.

Maintenance

Before saying anything about maintenance it is wise to define this term.
Maintenance signifies:

> All activities aimed at retaining an object's technical state or at reverting it back to this state, which is considered a necessary condition for the object to carry out its function.

These activities include both the repair of the structural strength, back to the starting level, and any inspections.
The costs of maintenance of civil engineering structures amounts to approximately 1% of the founding costs per year. For a life-span of 100 years this means that the maintenance costs are of the same magnitude as the construction costs. Taking into account the decline in new housing development projects, maintenance costs are clearly becoming an increasingly greater share of the expenses. A direct consequence is the desire to minimise maintenance costs. In order to realise this, the optimal maintenance strategy has to be sought. From the mechanical engineering maintenance theory, the following classification of strategies is known:

1. Curative maintenance - fault dependent maintenance
2. Preventive maintenance - use dependent maintenance
   - condition dependent maintenance.

In the case of fault dependent maintenance, an object is repaired or replaced when it can no longer fulfil its function (Figure 8). Thus, repair takes place after failure, therefore a failure norm is involved. The life of an object is fully exploited. An object's failure (and the associated costs) are accepted.
In hydraulic engineering this type of maintenance is usually not acceptable because, generally, the accepted probability of failure is limited. This type of maintenance can, however, be applied to non-integral construction parts (parts which do not contribute to the stability of the entire structure), with modest consequences of failure (provided reparation or replacement is not postponed for too long).

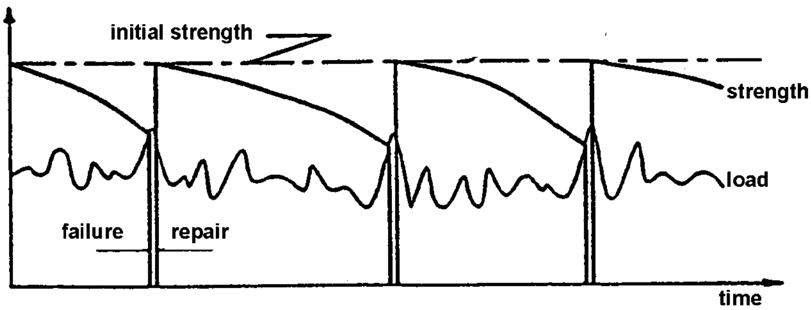

Figure 8. Possible course of strength with fault dependent maintenance

In the case of use dependent maintenance, maintenance is carried out after a period consisting of a previously determined number of usage units. The costs of maintenance and of the risk generally determine the extent of this period. The life of the object is not totally exploited. In mechanical engineering, this type of maintenance is applied if the usage units can be registered, for example with a kilometre indicator, product counter, etc..
In hydraulic engineering, this is different. We can hardly register all loads for every construction to organise the maintenance around those registrations. In this case, the loads in a period are considered random variables. Subsequently, the life time is estimated and the time for repairs is determined for which

the probability of failure is sufficiently small and for which the costs are minimal. Therefore, one should use the term time dependent maintenance (Figure 9). A time norm is involved.

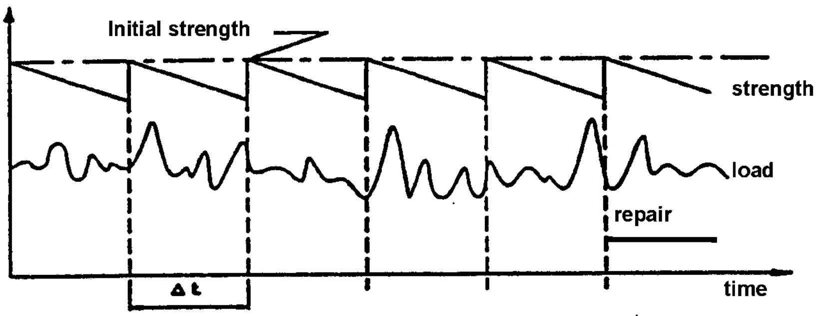

Figure 9. Possible course of the strength for time dependent maintenance

If loads which cause deterioration are registered, maintenance (inspection or repair) can be called for after an extreme large load or after a certain total amount of loading (cumulative). This is classified as load dependent maintenance, involving a load norm. In case the cumulative load plays a role, the possible courses of strength and load are given in Figure 10.

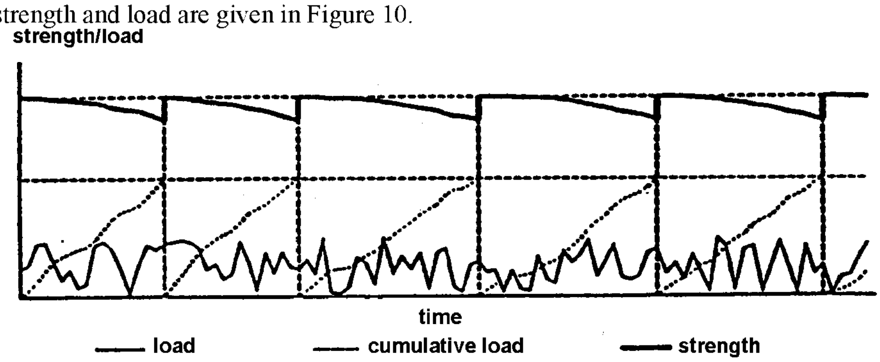

Figure 10. Possible course of the strength for load dependent maintenance

With condition dependent maintenance the state of the object is determined at set intervals, by means of inspections. The decision whether or not to carry out repairs is based on observations. The inspection intervals can be regular or dependent on the condition of the object. In the latter case condition parameters, indicating the condition of the object, have to be visible. The probability of failure in a period between two inspections has to be sufficiently small. Generally, the life time of the object can be better exploited than with usage dependent maintenance, but the costs of the inspections do have to be taken into account.
This type of maintenance also involves drawing up norms.

These norms concern (Figure 11):
1. a limit state which leads to an increase of the inspection frequency (warning threshold)
2. a limit state which leads to carrying out repair works (action threshold).
In fact this concerns strength norms. These norms result from an optimisation of the maintenance or correspond to a socially accepted failure probability in a year.

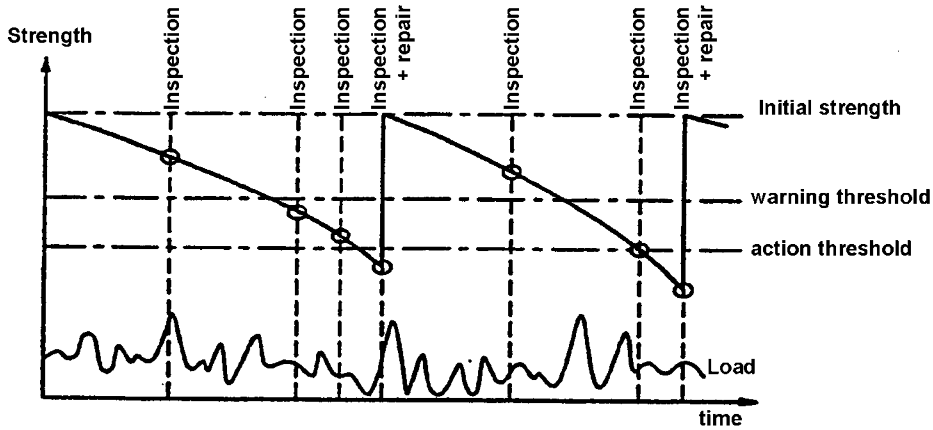

Figure 11. Possible course of strength with condition dependent maintenance

The selection of the maintenance strategy to be used depends on factors such as:
- predictability of the life-span of the object;
- consequence of failure of the object;
- costs of replacement or repair;
- costs of inspection;
- visibility of the condition of the structure (damages or deterioration).

A first comparison of the different strategies, focussing on the usability is presented in Figure 12.

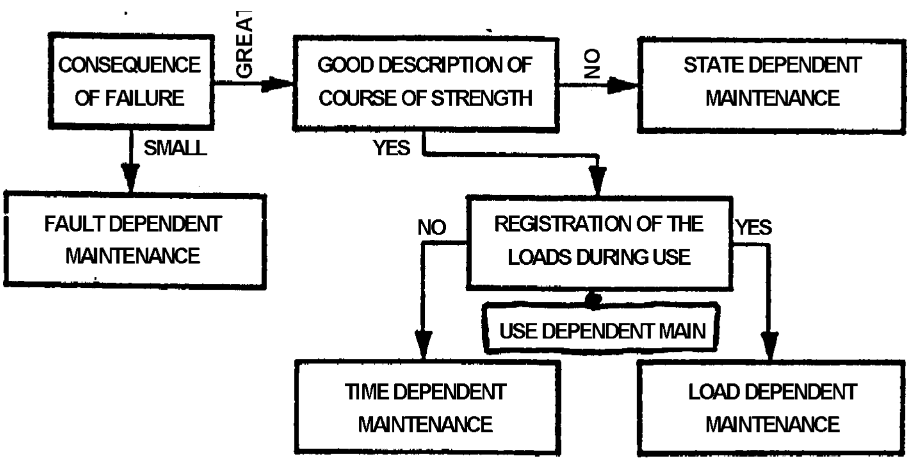

Figure 12. Global selection of the maintenance strategy

In hydraulic engineering a combination of two or more strategies appears to lead to a better result than simply applying a strategy selected using Figure 12. Thus, for example, a planning can be drawn up using the time dependent maintenance strategy, which can be adapted on grounds of observed loads, whilst the initiative to commence repair works is dependent on the strength according to inspection. Based on this example, one can state that the limits of applicability of the different strategies are flexible for hydraulic engineering.

Generally, however, statements can be made about the consequences of failure and with that the acceptance of fault dependent maintenance. Choosing between time dependent and condition dependent maintenance is less simple. Completely time dependent maintenance will be applied if inspection is not possible or if inspection is expensive relative to repairs. Completely condition dependent maintenance will be carried out if it is not possible to make a prognosis of the strength in the course of time or if inspection is very simple and therefore inexpensive.

An important aspect of the condition dependent maintenance is collecting data concerning the strength in the course of time. This allows for better planning of the maintenance or the inspections.

This paper will pay no further attention to the purely condition dependent maintenance, which requires no further knowledge of the wear of a construction.
The aim of this paper is to give a method which can be used to determine an economically sound planning of the times of repair and/or inspection of a structure.
Existing design models will be used for the deterioration of the structure.
The expected value of the costs of repair, inspection and the risk are central. To determine these expected values of the costs, a grasp of basic statistical techniques is necessary.

Deterioration models

The relation between strength and time is given by a deterioration model. The relation can be linear, exponential, logarithmic, etc.. In the case of development of the compression strength of cement stone, an asymptotic relation clearly exists (Figure 13). After approximately 30 days, the compression strength of the cement stone almost equals the maximum value.
Settlement of a dyke on a thick, hardly permeable layer also involves a limit value of the strength which can be observed. The settlement approaches a final value (Figure 14).

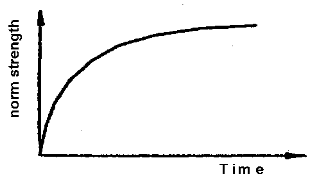

Figure 13. Course of the compression strength of cement stone

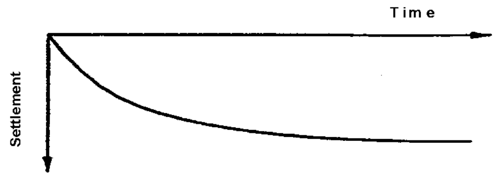

Figure 14. Course of settlement in time (consolidation)

The deterioration model thus determines the strength at every point in time. The model is an approximation of reality. The input required for the model is the starting strength and usually a number of parameters which describe characteristics of the material or the structure.
The parameters which serve as the input for the deterioration model are usually determined from tests or observations. They rarely have a certain value and can usually be best described by a random variable. This means that the strength at a certain time is a function of random variables and is thus a random variable itself.

Life-span of a structure without maintenance

The life-span of a structure is the time which passes between the realisation of the structure and the failure of the structure. In Figure 15 this is clearly marked for time dependent strength and load, for which the exact values are known for every point in time. The intersection of the strength and the load determines the moment of failure of the structure.

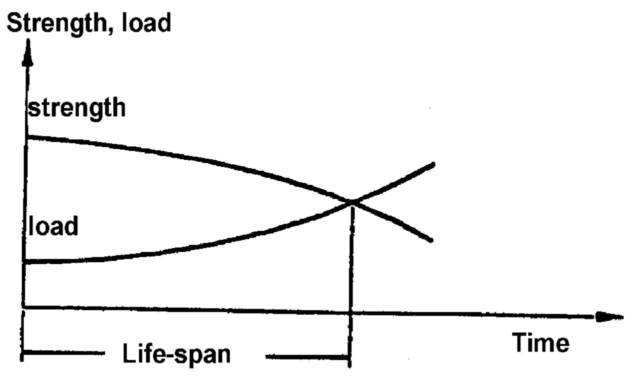

Figure 15. Life-span for an exactly known course of strength and load in time

In the case above, both strength and load are deterministic. The life-span is simple to determine, in that case. This is less so, when the load and/or the strength are random variables because, then, the life-span is also a random variable. The definition of the probability distribution of the life-span is:

$$F_L(t) = \Pr\{L < t\} = 1 - \Pr\{R > S \textit{ for every } \tau \textit{ in the interval } (0,t)\}$$

For the consideration of $\Pr\{R > S \textit{ in the interval } (0,t)\}$ the load has to be defined as the dominating load in the period (0,t). As t increases the average of the load in the interval (0,t) will also increase. This way the dependence on the duration of the dominating load is incorporated in the probability distribution of the load. If the strength is also time dependent a new problem arises, namely the determination of the normative strength for the period (0,t).

Conclusions

The new probabilistic approach has great advantages compared with the present. The event that the system of water defences is meant to prevent (flooding), comes at the center of the analysis. The contribution of all elements of the system and of all failure mechanisms of each element to the probability of flooding is calculated and clearly presented. The possibility to include the probability of human failure in the management of water defence structures is especially attractive and useful. The length effect, meaning that a longer chain is likely to have a weaker link, is adequately accounted for in the new method.

The results of the application of the method to Central Holland reveal indeed a ranking of the weaker elements. A plan to invest increasing amounts of money by progressively improving the elements can be defined on the basis of the analysis. At this moment the optimal inspection and maintenance of the dike system amalgamates into a far sighted plan to improve the safety of the entire system. Also a wider scale of measures becomes eligible for decision making with the new method than before. Investment in inspection, in research and in adapting spatial plans are alternatives that can be compared with the classical measures of dike improvement.

Finally an approach is sketched to define the level of acceptable risk. The decision on the level of acceptable risk is a cost/benefit judgement, that must be made from individual as well as from societal point of view. A system of three rules is developed to support the decision how safe the dikes should be. The most stringent of the three criteria should be adopted as a basis for the "technical" advice to the political decision process. However all information of the risk assessment should be available in the political process. A decision that is political in nature, must be made democratically, because many differing values have to be weighed. The economic optimisation may however show that the economic activity in the protected areas has grown so much since the 1950s that a fundamental reassessment of the acceptability of the flood risks is justified. Moreover the image of the Netherlands as a safe country to live, work, and invest in is an important factor to consider especially when ever more ambitious private and public investments, particularly in infrastructure, are planned.

References

Ministry of Housing, Land Use Planning and Environment, (1988). *Dutch NationalEnvironmental Plan.* The Hague, the Netherlands.

Health and Safety Executive, (1989). *Risk criteria for land-use planning in the vicinity of major industrial hazards.* HM Stationary Office.

Technical Advisory committee on Water retaining structures (TAW), (1984). *Some considerations on acceptable risk in the Netherlands.* Dienst Weg- en Waterbouwkunde (DWW), Delft.

Ministry of Housing, Land Use Planning and Environment, (1985). *LPG integral study* (in Dutch). The Hague, the Netherlands.

CUR, (1988). *Probabilistic design of flood defences*. Gouda, the Netherlands.

Vrijling, J.K., Wessels, J.F.M., van Hengel, W., Houben, R.J., (1993). *What is acceptable risk*. Delft University of Technology and Bouwdienst Rijkswaterstaat, Delft, the Netherlands.

Van de Kreeke, J., Paape, A. *On the optimum breakwater design*. Proc. 9th IC Coastal Engineering.

Dantzig, V.D., Kriens, J., (1960). *The economic decision problem of safeguarding the Netherlands against floods*. Report of Delta Commission, part 3, section II.2 (in Dutch), The Hague, the Netherlands.

Dantzig, V.D., (1956). *Economic decision problems for flood prevention*. Econometrica 24, New Haven, pp. 276-287.

Starr, C., (1969). *Social benefit versus technological risk*. Science, Vol. 165, pp. 1232-1283.

Ministry of Housing, Land Use Planning and Environment, (1992). *Relating to risks* (in Dutch). The Hague, the Netherlands.

Institution of Chemical Engineers, Engineering Practice Committee (1985). *Nomenclature for hazard and risk assessment in the in the process industries*. ISBN 0- 85295-184-1.

National Aerospace Laboratory (NLR), (1993). *Analyse van de externe veiligheid rond Schiphol*. CR 93485 L, Amsterdam, Nederland.

Vrijling, J.K., van Hengel, W., Houben, R.J., (1995). *Framework for risk evaluation*. Journal Hazardous Materials 43.

Vrijling, J.K., van Hengel, W., Houben, R.J., (1998). *Acceptable risk as a basis for design*. Journal Rel. Engineering and System Safety 59.

Vrouwenvelder, A., Vrijling, J.K., (1995). *Normstelling acceptabel risiconiveau*. PM-95-29.

Accounting for Dependencies in Inspection Planning for Steel Structures

D. Straub

Institute of Structural Engineering (IBK), Swiss Federal Institute of Technology (ETH)
e-mail: straub@ibk.baug.ethz.ch

Abstract

An overview is given on recent advances in Risk Based Inspection Planning (RBI) of structural systems. Inspection planning for systems must consider the effect of dependencies between the structural performance at the different locations in the system. It is shown how different methodologies, dependent on the applied mitigation strategy, can account for these dependencies. Adaptive strategies to inspection planning are introduced and justified, then two recently developed approaches to RBI for systems, based on these strategies, are reviewed and necessary further research and development is outlined.

Introduction

Risk Based Inspection Planning (RBI) has become a topic of large interest for engineering facilities in view of the increasing demands on the safety of the installations by the society (and therefore by legal authorities). A large number of publications have addressed the subject with respect to applications in different industries. The presented approaches are ranging from fully qualitative to fully quantitative approaches for maintenance planning and optimisation.

Quantitative approaches to RBI based on structural reliability analysis (SRA) and quantitative risk analysis date back to as early as Tang (1973) and have been developed mainly during the last 20 years, especially in offshore engineering, see e.g. Skjong (1985), Madsen et al. (1989) and Fujita et al. (1989). In an accompanying paper Faber (2003) presents a state-of-the-art methodology that, due to the introduced simplifications, has proved to be well suited for application in practice. Figure 1 shows a typical output of such a RBI procedure for individual elements.

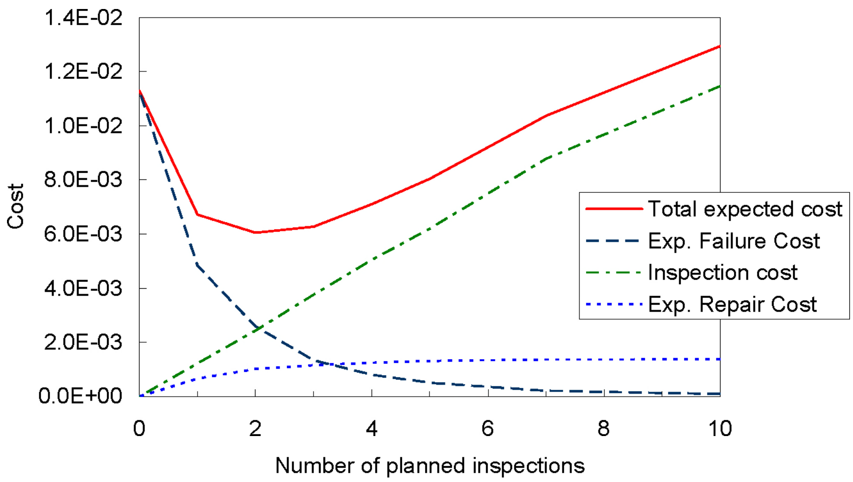

Figure 1. Inspection optimization of single elements from Straub and Faber (2002a)

While quantitative RBI on a element-by-element basis has reached the stage of practical applicability, quantitative inspection planning procedures considering entire systems have been the subject of only a small number of publications. The effect of dependency between individual elements on the updating after the inspection (posterior analysis) has been investigated in some studies, see e.g. Moan and Song (1998), Cramer and Friis-Hansen (1994), Faber and Sorensen (1999), Lotsberg et al. (1999) as well as Straub and Faber (2002a). In Faber et al. (1992) an informal decision analysis is proposed where the number of considered elements is reduced in a consistent and systematic way. However, due to the required numerical effort and stability, such approaches have not been demonstrated to be practical. Even though its importance is emphasized in many publications, an approach to RBI of systems that is directly applicable in practice has not been described to date.

In this paper recent advances in RBI of systems are reviewed. Although these approaches have not yet reached the state of industrial applicability, they represent a first basis for the consistent treatment of inspections of systems within the framework of pre-posterior analysis of the Bayesian decision theory. Due to dependencies in the system, information is obtained not only on the inspected element itself, but also on all other elements. These dependencies increase the effect of inspections on the perceived system performance and must thus be considered by the inspection optimisation procedure. The methodologies reviewed in this paper account for this effect when optimising the inspection effort.

System model

For the purpose of integrity assessment and modeling it is convenient to represent the system by discrete (finite) elements. In most cases the largest defect in each element is of interest, modeled by the random variable S_i, the size of the largest defect in element i. The S_i are described by the marginal distribution functions $F_{S_i}(s_i), i = 1,2,..,N$ and the covariance matrix, $COV_{\mathbf{SS}}$, that represents the stochastic dependency between the largest defects of the individual elements. Typical examples of such elements in real systems are e.g. hot spots in a steel structure subject to fatigue failures or finite elements of a structure subject to corrosion. The model is illustrated in Figure 2.

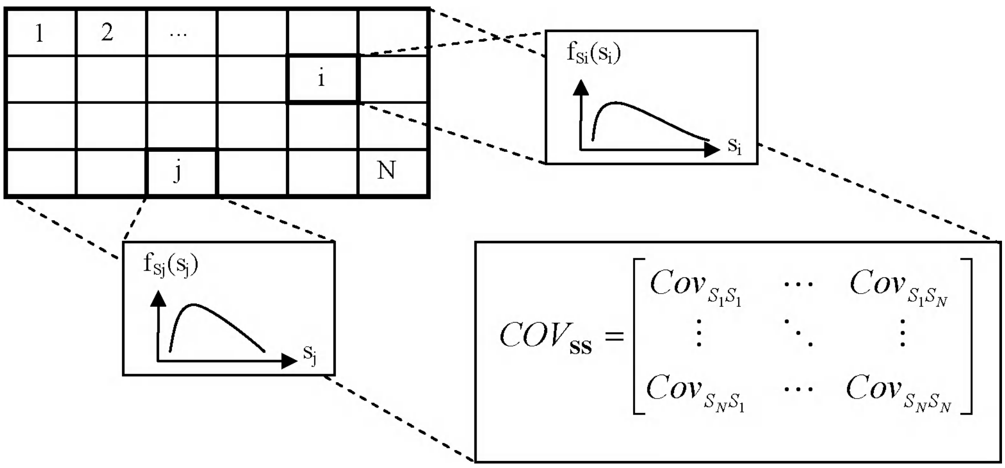

Figure 2. Illustration of the system deterioration model from Straub and Faber (2002b)

In principle the individual $F_{S_i}(s_i)$ can be different for all elements. For many systems the model of the deterioration will, however, be identical for a large number of elements. E.g. for a pipeline with homogenous conditions along its axis the prior model of corrosion depth will be the same for all elements along the bottom.

For deteriorating systems time is an additional dimension that has to be considered. Defects can then be described by the marginal distribution functions $F_{S_{i,t}}(s_{i,t})$ of the largest defect in element i at time t and the corresponding covariance matrix. Mostly a more sophisticated model is applied where the deterioration process under consideration is modelled explicitly; the spatial model however remains as illustrated in Figure 2.

Overview on modelling and planning inspection of systems

RBI for time invariant problems (e.g. defects that are constant with time) reduces to the problem of determining where to inspect, what type of inspections to perform and what to look for. There are generally limited numbers of possible combinations of these parameters. On the other hand, if the inspection effort is optimised for deteriorating structures, additionally the times of the inspections can vary, which makes RBI for time variant problems much more complex and demands for corresponding procedures.
For systems, in accordance with Straub and Faber (2002a & b), a second distinction is made between cases where common mitigation actions for all elements are applied and cases where the individual elements are treated separately. For concrete structures repairs are often planned and performed for an entire systems or sub-system (a typical system can be an entire concrete wall subject to corrosion). The repairs to perform are decided for the entire system based on sample inspections which are assumed to represent the overall system with sufficient accuracy. For steel structures, such strategies can be reasonable for pipelines where repairs are difficult to perform for individual parts only. On the other hand structures subject to fatigue are typically systems with discrete elements (namely the hot spots subject to fatigue). These are generally assessed and repaired individually, i.e. each repair decision is based on previous inspection of the hot spot under consideration. The decision process is thus more complex for these systems because, as demonstrated in Straub and Faber (2002b), the correlation between individual hot spots can only be accounted for by applying an adaptive inspection planning strategy as presented below.
The four principally different cases are illustrated in Figure 3 together with the respective inspection optimization procedures.

	Common mitigation actions	**Individual mitigation actions**
Time invariant	(a) Classical Bayesian quality control theory	(c) System POD
Time variant (deterioration)	(b) Condition indicator methodology	(d) Modular approach

Figure 3. Four different solution strategies to the inspection planning for systems

Case (a) & (b) - Common mitigation actions

For systems consisting of elements subject to identical conditions and for which preventive and corrective maintenance actions are the same for all elements, i.e. when all elements are replaced or repaired given that a certain percentage of the elements have reached an unacceptable state, the inspection and maintenance planning problem reduces to a classical quality control problem. For time invariant problems the optimization of testing (inspection) is straightforward and can be performed according to the classical references of e.g. Raiffa and Schlaifer (1961) or Benjamin and Cornell (1970). Wall and Wedgwood (1998) show a (simplified) application of the approach, where they determine the cost optimal inspection coverage. Considering deteriorating systems Faber and Sorensen (2002) extend the classical solution and introduce so-called condition indicators. The indicators (which in principle are inspection results) give information about the state of the overall structure at different point in times. By evaluating the content of these information at different times and for different inspection coverages, the optimal number and time of inspections may be evaluated.

Case (c) & (d) – Individual mitigation actions: Adaptive strategies

For systems where repairs are only planned after a previous inspection, inspection of dependent elements has no direct influence on the repair decision of the non-inspected elements. The non-inspected elements will not be repaired. However, if only part of the system is inspected, based on the result of the inspections additional inspections may be planned. This is an adaptive strategy. Adaptive inspection strategies have the characteristics that the total number of inspections is not known beforehand but depends on the performance of the structure. The inspections are thus more targeted towards systems that perform worse than average. In the following section it will be shown how these adaptive strategies can be formulated and optimized based on Straub and Faber (2002a & b).

Inspection planning for structural steel systems subject to fatigue

Time invariant case

If non-propagating cracks are considered, then the methodology as presented by Straub and Faber (2002b) is directly applicable. It is based on the idea of evaluating the expected number of detected cracks in the system. Because this approach is in analogy to the classical Probability of Detection (*PoD*) concept for individual hot spots, it is denoted the *System PoD* (PoD_S). It is demonstrated in Straub and Faber (2002b) that the PoD_S not only depends on the quality of the inspection, but also on the correlation structure of the cracks in the system and the applied adaptive inspection strategy. It is thus a common characteristic of the system and the inspection method.

The adaptive inspection strategy for a system of N elements is formulated as follows: In a first round, the γ^{th} fraction of all hot spots are inspected, i.e. $\gamma \cdot N$ elements. If more than the q^{th} fraction of these inspections result in detection (i.e. $\gamma \cdot q \cdot N$ elements), than all hot spots are inspected, otherwise no additional inspections are performed. This strategy can be refined as long as its adaptive character is maintained. Figure 4 shows the PoD_S for the defect and inspection model from Straub and Faber (2002b) when q is equal to 0.3. Clearly the PoD_S depends on the dependency structure within the system (the correlation between the individual hot spots), expressed by the correlation factor r. Increasing correlation increases the amount of information on the non-inspected hot spots as gained by the first inspections and so increases the PoD_S. A non-adaptive strategy (no additional inspections regardless of the outcome) leads to a linear PoD_S.

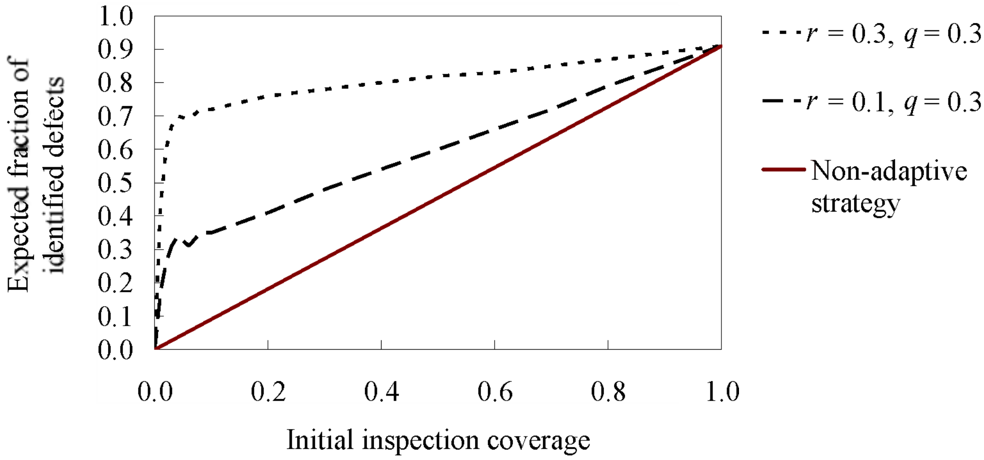

Figure 4. System PoD for different correlation between the individual elements

The parameters γ and q, as well as the inspection coverage, can be optimized with respect to the total expected costs when the expected cost of missing a defect is quantified. Such an optimization is illustrated in Figure 5.

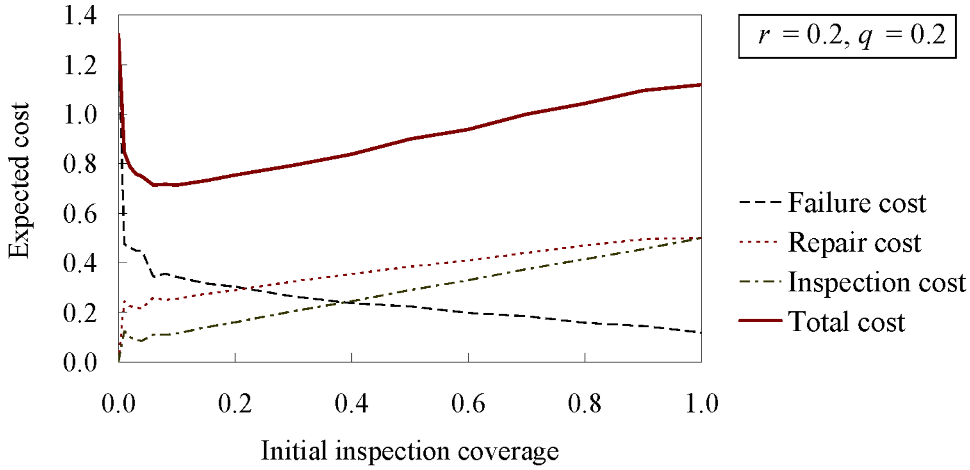

Figure 5. Inspection optimisation based on the system PoD concept from Straub and Faber (2002b)

Time variant case

If time is considered in the inspection planning the adaptive strategy becomes more complex. In principle an infinite number of possible combinations of inspection times and locations exists and optimisation becomes very difficult. This is discussed in Straub and Faber (2002a). Based on the generic approach to RBI of single hot spots, see Faber (2003), a methodology for the RBI of systems is proposed by Straub and Faber (2002a) to overcome the problem. Their basic idea is to relate the performance of a hot spot to the (deterministic) fatigue design factor *FDF* which is obtained from design calculations. This allows to calculate the expected value of information (on the system) gained by an inspection (the value of the inspection comes from the fact that it reduces the expected failure costs). This is based on the value of information concept from the classical Bayesian decision theory, see Raiffa and Schlaifer (1961).
Using the generic approach, it is possible to determine the optimal inspection plan and the total expected cost for each individual hot spot as a function of the *FDF*. Given the inspection results, the *FDF*'s of the non-inspected hot spots are updated. Because each *FDF* implies an optimal inspection strategy, the inspection times of the non-inspected hot spots are automatically adjusted with the updated *FDF*. The methodology in this way forms an adaptive inspection strategy.
Figure 6 shows the value of information that is obtained by inspection of one hot spots as a function of the *FDF* of the inspected hot spot and the time of inspection. It is the expected amount of obtained information for a dependent[1] hot spot.

[1] Simplifying it is assumed that the stress ranges are fully correlated from one hot spot to the other, all other random variables in the deterioration model are taken as independent.

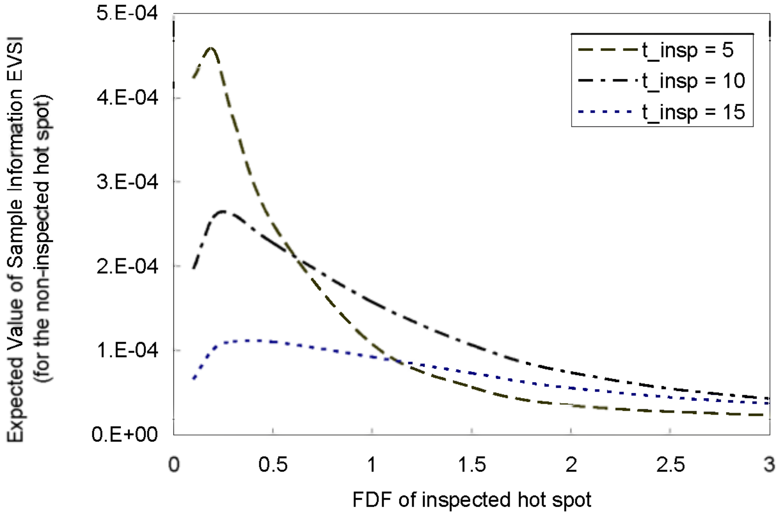

Figure 6. Value of information gained from inspection of a dependent hot spot for different times of inspection (5 to 15 years), from Straub and Faber (2002a)

By calculating the value of information from a set of inspections, the optimal numbers and times of inspections can be evaluated. Such an example is illustrated in Figure 7. The amount of information converges with increasing number of inspections toward a maximum value (the expected value of perfect information).

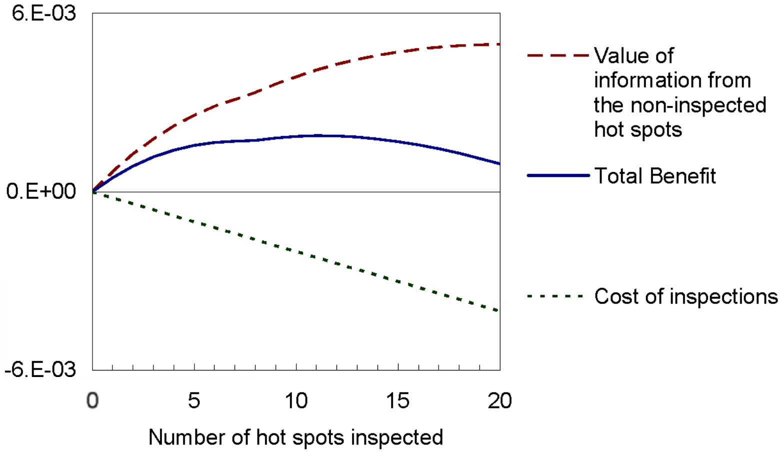

Figure 7. Optimisation of the number of inspections at a certain time, considering the effect on the dependent hot spots, from Straub and Faber (2002a)

Conclusion and discussion

Two different approaches to RBI for systems based on adaptive inspection strategies were reviewed. Their applicability is still restricted. The system PoD approach is to date only applicable to non-deteriorating defects, e.g. the optimisation of initial inspections or inspections at a given point in time. The modular approach, based on the generic approach to RBI, is more sophisticated and allows for a consistent incorporation of the system effects (due to dependencies between the individual hot spots or elements) in the inspection planning procedure. Unfortunately, the methodology in its present form is too complex for application in practice.
Nevertheless, the presented methods can be used to assess the influence of dependencies in the system and may be applied in a semi-quantitative manner to account for system effects in the final inspection scheduling (e.g. by reducing the number of similar inspections when no cracks are found). In addition,

further development could allow to extend the system *PoD* concept so that it is applicable in a consistent manner also for time varying defects.
Very little is known on dependencies between the resistance against deterioration at different locations in the structures for all types of deterioration mechanisms. Further research is thus needed mainly on the understanding and modelling of the dependency structure in deteriorating structural systems.

References

Benjamin JR, Cornell CA (1970). *Probability, statistics and decision for civil engineers*. McGraw-Hill, NY.

Cramer EH, Friis-Hansen P (1994). *Reliability-Based Optimization of Multi-Component Welded Structures*. Journal of Offshore Mechanics and Arctic Engineering, 116, pp. 233 - 238

Faber MH (2003). *Generic Inspection Plans for Steel Structures Subject to Fatigue*. Proc. Workshop on Risk Based Maintenance, TU Delft, Netherlands

Faber MH, Sørensen JD (1999). *Aspects of Inspection Planning – Quality and Quantity*. Proc. ICASP8, Sydney, pp. 739-746

Faber MH, Sørensen JD (2002). *Indicators for inspection and maintenance planning of concrete structures*. Structural Safety, 24(4), pp. 377 - 396

Faber MH, Sørensen JD, Kroon IB (1992). *Optimal Inspection Strategies for Offshore Structural Systems*. Proc. 11th Offshore Mechanics and Arctic Engineering Conference, Calgary, pp. 145-151

Fujita M, Schall G, Rackwitz R (1989). *Adaptive Reliability-Based Inspection Strategies for Structures Subject to Fatigue*. Proc. 5th ICOSSAR, Vol. 2, pp. 1619-1626

Lotsberg I, Sigurdsson G, Wold PT (1999*). Probabilistic Inspection Planning of the Åsgard A FPSO Hull Structure with Respect to Fatigue*. Proc. 18th Offshore Mechanics and Arctic Engineering Conference, New Foundland, Canada, paper S&R-6040, pp. 259-266

Madsen HO, Sørensen JD, Olesen R (1989*). Optimal Inspection Planning for Fatigue Damage of Offshore Structures*. Proc. 5th ICOSSAR, Vol. 3, pp. 2099-2106

Moan T, Song R (1998). *Implication of Inspection Updating on System Fatigue Reliability of Offshore Structures*. Proc. 17th Offshore Mechanics and Arctic Engineering Conference, paper S&R-1214

Raiffa H, Schlaifer R (1961). *Applied Statistical Decision Theory*. Cambridge University Press, Cambridge, Mass.

Skjong R (1985). *Reliability Based Optimization of Inspection Strategies*. Proc. 4th ICOSSAR, Vol. 3, Kobe, Japan

Straub D, Faber MH (2002a). *System Effects in Generic Risk Based Inspection Planning*. Proc. 21st Offshore Mechanics and Arctic Engineering Conference, Oslo, paper S&R-28426.

Straub D, Faber MH (2002b). *On the Relation between Inspection Quality and Quantity*. Proc. European-American Workshop on Reliability of NDE, Berlin

Tang WH (1973). *Probabilistic Updating of Flaw Information.* Journal of Testing and Evaluation, 1(6), pp. 459-467

Wall M, Wedgwood FA (1998), *Economic Assessment of Inspection – the Inspection Value Method.* The e-Journal of Nondestructive Testing, 3(12), available online: www.ndt.net/journal/archive.htm

Condition indicators for inspection and maintenance planning

R. Wicki[1], V. Malioka[2] & M.H. Faber[3]

Swiss Federal Institute of Technology, Zurich, Switzerland

[1]Diploma Thesis Student [2]Research Assistant [3]Professor Dr.

e-mail [1]rwicki@student.ethz.ch [2]malioka@ibk.baug.ethz.ch [3]faber@ibk.baug.ethz.ch

Abstract

The identification of cost efficient maintenance strategies for concrete structures usually takes basis in condition assessments achieved through inspections, tests and monitoring. It is well recognized that such condition assessments are subject to significant uncertainties and in general at best provide indications rather than observations about the condition of the structure. The present work proposes a probabilistic framework for the quantification of inspection results for concrete structures in regard to their actual condition, the predicted future degradation, the estimated remaining service life and the expected service life costs of the structure. Within this context it is shown how condition indicators, based on non-destructive evaluation methods, can be formulated to enable quality control and inspection and maintenance planning. Furthermore it is investigated how the "strength" of the indicators not only depends on the inspection method but moreover may be significantly dependent on the age of the structure at the time of the inspection.

Introduction

As a consequence of the degradation processes such as e.g. ingress of chlorides and subsequent corrosion of the reinforcement, concrete structures will degrade gradually and if no intervention is made the state of the structures may become a matter of significant risk, if not to personnel then in terms of costs. The economical efforts required to maintain the already existing stock of concrete structures are rapidly increasing and consequently engineers are looking for ways to balance the risks associated with ageing structures against an inevitable shortage of funds for maintenance and repair.

Regular inspections form an important tool in condition control of concrete structures and the information gathered should form the basis for repair and maintenance planning. In practice the quantification of inspection results is highly person dependent (subjective) and therefore the overall assessment of the actual state and future performance of concrete structures is not performed in a rational and systematic way. To overcome this problem recent work has been aiming to identify consistent approaches to the quantification of inspection results for the inspection methods commonly applied in practice for the condition assessment of concrete structures. Benjamin and Cornell (1970) introduced condition indicators in a Bayesian framework for quality control. In Faber and Sorensen (2000) and Faber and Sorensen (2002) this idea has been elaborated further and it is shown that the quality of half-cell potential measurements may be consistently quantified for assessment purposes in terms of the effect of the measurement results on the predicted life time of the structure. This both for structures with localized deterioration and for structures with spatially distributed deterioration. In Lentz et al. (2001) a systematic study is reported regarding the probabilistic modeling of the quality of half-cell potential measurements as a means of condition control for highway concrete bridges exposed to thaw salt. There it is also shown how the probabilistic models of the half-cell potential measurements may adequately be categorized in accordance with exposure class (chlorides and humidity) and that the limiting potential in fact is a decision variable to be optimized under consideration of the service life costs of the structure. However, half-cell potential measurements is only one alternative among several different inspection methods applied as a means for the regular condition control of structures. General visual inspections, cover meter measurements, chloride profiles, Torrent

porosity measurements, see e.g. Torrent (1992) and built-in corrosion cells represent a selection of other methods used in practice and for which the quality quantifications is still to be established.
In the present paper three of these indicators are considered, namely macrocell systems, the cover thickness measurements and carbonation depth measurements using phenolphthalein. Limit state functions for the events defining new information about the structure are formulated and it is demonstrated how these are applied to update the probability of adverse events such as initiated or visual corrosion. Two features of the indicators characterizing the relevance and the quality of the indicator as a means for condition control are then studied in detail, namely the strength of the indicator, i.e. the probability of the (adverse) state of interest conditional upon a certain indication and the likelihood of the indicator, i.e. the probability of observing the indication conditional upon the (adverse) state of interest.

Degradation and Condition Control

Following e.g. Faber (1997) inspection and maintenance strategies can be optimized by consideration of their effect on the expected service life benefits. Optimal inspection and maintenance strategies may thus normally be identified as those minimizing the expected total service life costs including the costs of inspection and testing, the costs due to future maintenance and repair and the costs due to future failures

$$E[C_T]=E[C_I]+E[C_R]+E[C_F] \tag{1}$$

where $E[C_I]$, $E[C_R]$, $E[C_F]$ are the expected inspection, repair and failure costs respectively. In probabilistic terms the expected costs can then be determined as the probability that the cost-inducing event will occur multiplied by the cost consequences given the event. However there are many aspects that the engineer needs to consider before the cost evaluation.
Regarding inspection and maintenance planning of concrete structures, the development in time of the most important deterioration mechanisms may be described by two main phases namely initiation and propagation. Each deterioration phase is associated with different characteristics and consequences. For the case of deterioration due to corrosion these phases and their characteristics may be seen in Figure 1, see also Faber and Gehlen (2002).

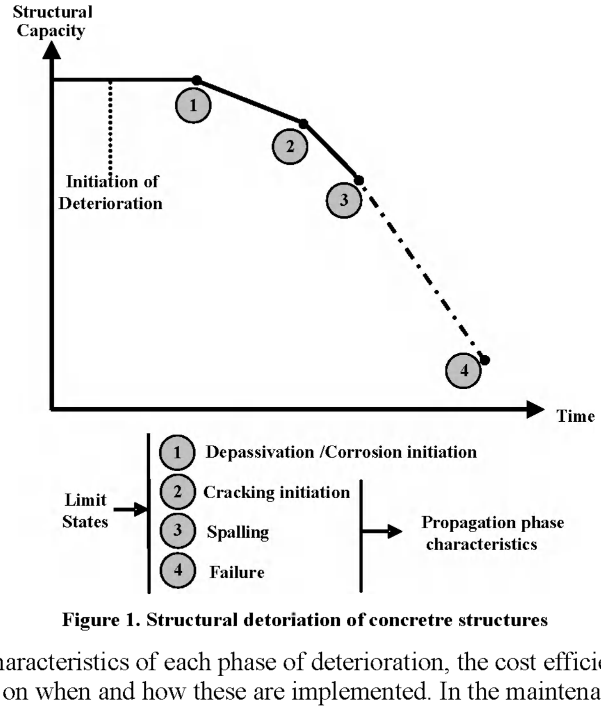

Figure 1. Structural detoriation of concretre structures

Based on the different characteristics of each phase of deterioration, the cost efficiency of maintenance and repair activities depends on when and how these are implemented. In the maintenance decision problem a valuable tool is the information gathered through inspections. To this end there are many directions that may be followed in regard to the type of the inspection and the time of execution of the inspection.
If the progress of deterioration is detected at an early stage, the risk of major and costly future repair works may be significantly reduced by relatively inexpensive preventive maintenance. The question, however,

remains in regard to which method of inspection is the more suitable and at what time during the service life of the structure it is more efficient to be implemented. Thus, referring to Equation 1, the minimization of inspection and repair costs is highly dependent on the type of inspection and the time of its implementation.

Bayesian approach to inspection and maintenance planning

Application of Bayesian decision theory to inspection data

Decision making or providing decision basis in problems influenced by uncertainties and/or lack of knowledge is a central role for the engineers. In addition to that when decisions are to be made within limited budgetary constraints the importance of the information in hand becomes even more significant. The main resource of information regarding the true state of a structure is on-site inspections from which information may be obtained to update the currently available knowledge. However it is not always feasible or economically efficient to collect additional information concerning the state of the structure. In addition to that the choice of the inspection method will affect the resulting decisions both technically and economically. Bayesian decision theory provides a mathematical model for making engineering decisions subject to uncertainties, Benjamin and Cornell (1970).

The concept of Bayesian statistics and its importance in knowledge updating for the purpose of reliability based assessment of structures is treated in detail elsewhere, see e.g. JCSS (2001). Here only the direct application for the present problem is addressed further.

The probability of a structure being in a certain state e.g. exhibiting visual corrosion $\{CV(t)\}$, at time t may be evaluated as

$$P(CV(t)) = P(g(\mathbf{X},t) \leq 0) \tag{2}$$

where X is the vector of random variables describing the degradation process under consideration and where $\{g(X,t) \leq 0\}$ in general is a limit state function defining the state of interest. When additional information becomes available e.g. through an inspection, the probability of the structure being at a certain state may be updated by use of the rule of Bayes. Thus for an inspection result described by the event $\{I\}$ the updated (posterior) probability of visual corrosion becomes

$$P''(CV(t)|I) = \frac{P(I|CV(t))P'(CV(t))}{P(I|CV(t))P(CV(t)) + P(I|\overline{CV}(t))P(\overline{CV}(t))} \tag{3}$$

where I is the inspection result - the indication.

The mixing between new and old information takes place through the prior probability, $P'(CV(t))$, and the sample likelihood $P(I|CV(t))$ i.e. the probability of obtaining the indication I given the true state of $\{CV(t)\}$. From Equation (3) it is seen that the posterior probability depends not only on the direct quality of the inspection method, i.e. the likelihood but also on the prior probability. This effectively means that the outcome to be expected from an inspection is highly influenced by the probability of the state of concern, an issue that will be discussed later.

In Figure 2 an example is given of the updating scheme based on the numerical evaluations of Equation (3).

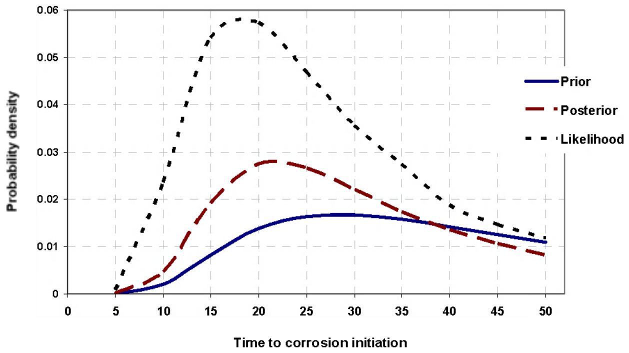

Figure 2. Illustration of the Bayesian updating scheme based on the application of Equation (3).

Condition indicators

Non-destructive evaluation (NDE) methods are less than perfect and there is always the probability of erroneous indications. Within the Bayesian decision framework condition indicators may be formulated for addressing the probability of false indications, for use in knowledge update as frequentistic information is collected and for a more consistent prediction of the structure's future condition.

As described in Faber and Sorensen (2000) the quality of inspection methods may be quantified by the probability that the considered inspection indicates degradation given that degradation is actually present, $P(I_D|D)$, and in addition to that the probability that the applied procedure indicates degradation given that the last is not actually present, $P\left(I_D|\overline{D}\right)$. I_D represent inspection results such as a half-cell potential measurement and D can refer to one of the deterioration phases (see e.g. Figure 1) such as corrosion initiation. However there is also the probability of observing a negative result. This is expressed as the probability of no indication of deterioration given deterioration, $P(I_{\overline{D}}|D)$, and the probability of having no indication given that deterioration is not present, $P(I_{\overline{D}}|\overline{D})$.

These probabilities may be used to predict the state of the structure at some point in time in the future. Considering for example deterioration due to corrosion of the reinforcement inspection and maintenance planning may be described by the event tree shown in Figure 3. The event of visual corrosion at the time of repair decision T is indicated as CV and $\overline{CV}$ is the event of no visual corrosion. I_D and $I_{\overline{D}}$ refer to a positive and a negative indication of corrosion respectively, while CI and $\overline{CI}$, denote the events of corrosion initiation and no corrosion initiation at the time of the inspection t_i.

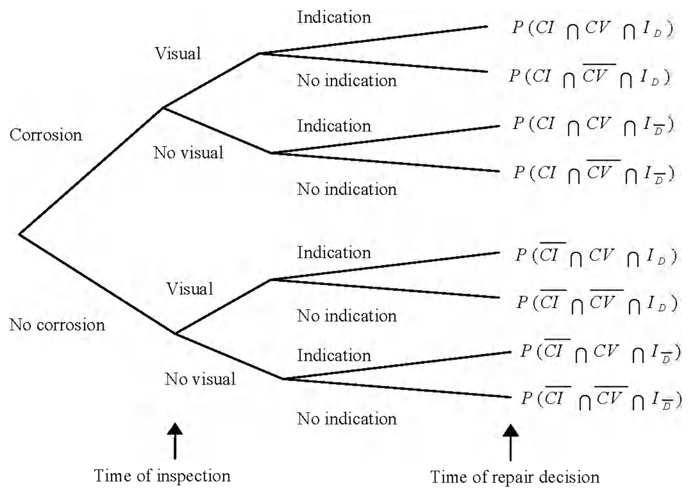

Figure 3. Event tree for condition states and inspection observations.

Based on the above event tree the probability of the structure showing signs of visual corrosion say after T =50 years given that a NDE has provided a negative indication of corrosion initiation at time t_i is given by

$$P(CV|I_{\overline{D}})=\frac{P(CV\cap CI)\cdot P(I_{\overline{D}}|CI)}{K_1}+\frac{P(CV\cap\overline{CI})\cdot P(I_{\overline{D}}|\overline{CI})}{K_1} \tag{4}$$

where

$$K_1 = P(I_{\overline{D}}|CI)\left[P(CI\cap CV)+P(CI\cap\overline{CV}\right]+ \\ P(I_{\overline{D}}|\overline{CI})\left[P(\overline{CI}\cap CV)+P(\overline{CI}\cap\overline{CV}\right] \tag{5}$$

And for the case of a positive indication the relevant probability is given by:

$$P(CV|I_D)=\frac{P(CV\cap CI)\cdot P(I_D|CI)}{K_2}+\frac{P(CV\cap\overline{CI})\cdot P(I_D|\overline{CI})}{K_2} \tag{6}$$

where

$$K_2 = P(I_D|CI)\left[P(CI\cap CV)+P(CI\cap\overline{CV}\right]+ \\ P(I_D|\overline{CI})\left[P(\overline{CI}\cap CV)+P(\overline{CI}\cap\overline{CV}\right] \tag{7}$$

The above probabilities may also be interpreted as the strength of the indicator, see also Faber and Sorensen (2002) where numerical studies of the half-cell potential measurement indicator are reported both for the case of localized corrosion as well as for spatially distributed corrosion.

Numerical Investigations

In accordance with the approach outlined above a series of numerical investigations of the quality of the cover meter, the corrosion cell and the carbonation depth measurements as indicators of the instantaneous as well as the future condition of concrete structures is given. The corrosion cell measurement provides direct information about the degradation phase of the structure at the time of the measurement i.e. whether or not corrosion initiation has taken place. The cover meter measurements only provide indirect information in regard to the degradation phase of the structure while the phenolphthalein measurements may supply information for the progress of deterioration due to carbonation. Furthermore, as shall be seen, all three methods may be applied as indicators of the future condition of the structure.

Probabilistic modelling

For the purpose of illustration a concrete structure is considered with the deterioration process being corrosion due to chloride ingress or carbonation. The two main deterioration phases have been examined i.e. corrosion initiation and propagation (see also Malioka and Faber (2003) and Wicki (2003)) while here illustrations are provided for the case of corrosion initiation . In the case of chloride attack corrosion initiation takes place when a certain critical concentration C_{CR} is exceeded. Assuming that the ingress of chlorides through the concrete cover may be described by a diffusion process with diffusion coefficient D the time till corrosion initiation at a depth z may be written as, DuraCrete (1999)

$$T_I = \frac{z^2}{4D}\left(erf^{-1}\left(1-\frac{C_{CR}}{C_S}\right)\right)^{-2} \qquad (8)$$

where C_S is the concentration of chlorides on the surface of the concrete.
In the case of corrosion due to carbonation, initiation is considered to occur when the carbonation front reaches the first layer of reinforcement. The time till initiation is given by the following expression, DuraCrete (1999)

$$T_I = \left(\frac{z^2}{2 \cdot k_c \cdot k_t \cdot k_e \cdot \frac{D}{a} \cdot C_S \cdot t^{2n}}\right)^{\frac{1}{1-2n}} \qquad (9)$$

where k_t is a test method variable, k_c is an execution variable, k_e and n are environmental variables while a represents the binding capacity for CO_2 and t is a reference period.
The time until corrosion becomes visual is expressed as the time till corrosion initiates plus a certain propagation time T_P i.e.

$$T_V = T_I + T_P \qquad (10)$$

The probabilities of corrosion initiation and visual corrosion may be estimated using e.g. FORM/SORM methods, Madsen et al. (1986), based on the following safety margins

$$M = g(X,t) = X_I \cdot T_I - t \qquad (11)$$

$$M = g(X,t) = X_I \cdot T_I + T_P - t \qquad (12)$$

where X_I is a model uncertainty associated with the corrosion initiation time. The probabilistic modelling of the parameters in the above equations may be found in Faber and Sorensen (2002) and Wicki (2003).

Macrocell techniques for corrosion monitoring

General

Information collected relative to ongoing corrosion of reinforcement in concrete is an important tool in assessing both the present and future performance of the structure. However it is not possible to remove the steel in concrete nor is it beneficial to remove the concrete cover for carrying out reinforcement checks. Macrocell techniques seem to be very promising as a means for non-destructive long-term monitoring of corrosion and the prediction of its future development. The method rests on embedding macrocell devices into the concrete. This includes embedding steel into the concrete, in the form of several steel bars at different depths, to create a corrosion cell in the shape of a ladder.
The current between the steel members is measured and provides an indication of corrosion initiation. A detailed description of the method can be found in Broomfield (1997) and Bentur (1997). For the evaluation of the strength of the indicator and the quality of the inspection the ladder and ring

configurations have been used as proposed in Schiessl and Raupach (1992) and Raupach and Schiessl (2001) (Figure 4).

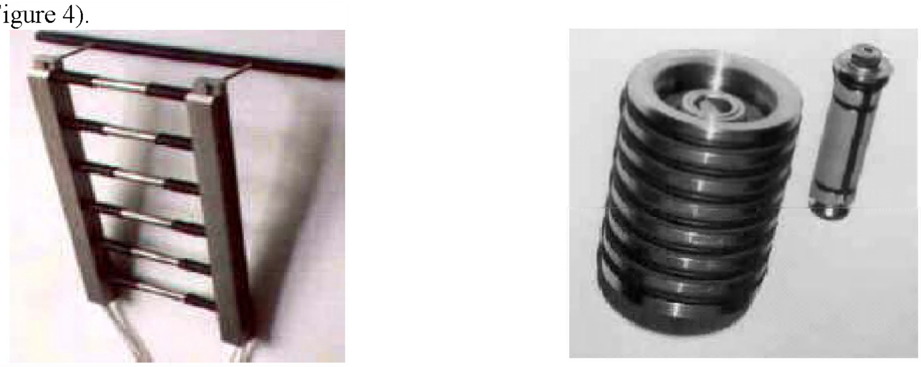

Figure 4. Sketch of the ladder macro-cell system (Broomfield (1997).

As the chloride or carbonation front reaches each steel bar this becomes active and the current passing from the anodes can be measured. Both configurations has been tested in a number of bridges, Schiessl and Raupach (1992) and Cusson (2002) and proved rather useful. Although there are some limitations relatively to the use of macrocell techniques, Broomfield (1997), they provide the advantage of facilitating long-term condition control and a means for checking the degradation state of the structure at any time.

Condition indication by a "ladder" macrocell system

In this case the indicator is the current passing through a steel bar located at a certain depth z . A rebar ladder is assumed with four steel rebars (rungs) at 20 mm increments for the examining chloride induced corrosion, while the ring configuration has been used for evaluating the indicator's performance for corrosion due to carbonation. The time until corrosion initiates at that depth is described by Equation (8). The limit state function for corrosion initiation at depth z is as in Equation (11) with $t = t_{insp}$ where t_{insp} is the time of inspection. According to the general approach of Bayesian updating the probabilities of corrosion initiation at the depth of reinforcement in a reference future time are estimated given a positive or a negative indication of current passing through a ladder bar. Equation (13) shows the posterior probability of corrosion initiation given that a current is passing through the system.

$$P\left(T_I - t \le 0 \middle| T_{I_z} - t_{insp} \le 0\right) = \frac{P\left(T_{I_z} - t_{insp} \le 0 \middle| T_I - t \le 0\right) \cdot P\left(T_I - t \le 0\right)}{P\left(T_{I_z} - t_{insp} \le 0\right)} \qquad (13)$$

For the case of "no indication" Equation (13) takes the following form

$$P\left(T_I - t \le 0 \middle| -(T_{I_z} - t_{insp} \le 0)\right) = \frac{P\left(T_{I_z} - t_{insp} \le 0 \middle| -(T_I - t \le 0)\right) \cdot P\left(T_I - t \le 0\right)}{P\left(-(T_{I_z} - t_{insp} \le 0)\right)} \qquad (14)$$

Accordingly the posterior probabilities for visual corrosion may be estimated using the limit state function shown in Equation (12), Malioka and Faber (2003).
Figures 5 and 6 provide an illustration of the probabilities of corrosion initiation in 50 years given a positive indication of corrosion initiation due to chloride ingress and carbonation respectively. As it can be seen (Figure 5) in the first 10 years the inspection using a ladder macrocell system will not provide much information regarding the initiation of corrosion at the depth of reinforcement, particularly for positions of the ladder rungs greater than 40 mm depth. That is because the probability of corrosion initiation is quite low until this point in time in such depths. Actually for a position of the rungs of 60 mm and more there is no differentiation in the results for at least up to 25 years. On the other hand for a depth of measurement of 20 mm the indicator may provide important information up to 15 to 20 years. After that the strength of the

indication is not that significant and it is of more interest to look at information gathered from a greater depth.

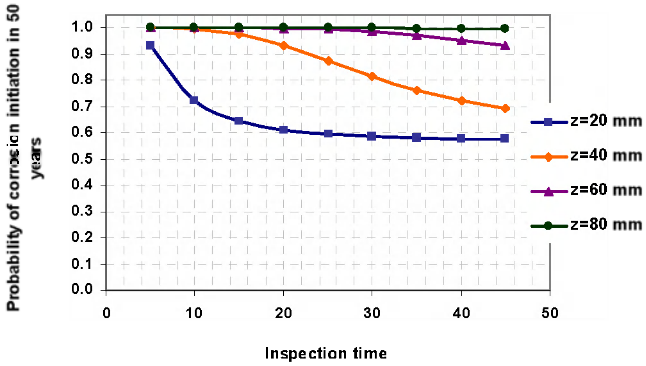

Figure 5. Probability of corrosion initiation at depth of reinforcement (t=50 years) given an indication of corrosion initiation at depth z (chloride model).

From Figure 6 it is seen that a positive indication of corrosion initiation at a depth of 10 mm in the first 5 years results in a high probability of corrosion initiation in 50 years. However for inspections after 20 years the indicator yields practically the same information in regards to the development of corrosion. Also this phenomenon appears for depths greater than 30 mm and inspections carried out earlier than 20 years. What is also interesting to look at is a comparison with the probabilities for corrosion due to chloride ingress. For example for a positive indication at a depth of 20 mm the probability of corrosion initiation falls rapidly in the carbonation model and that is due to the fact that carbonation is a slow process while in the chloride model the relevant probability remains at a quite high level.

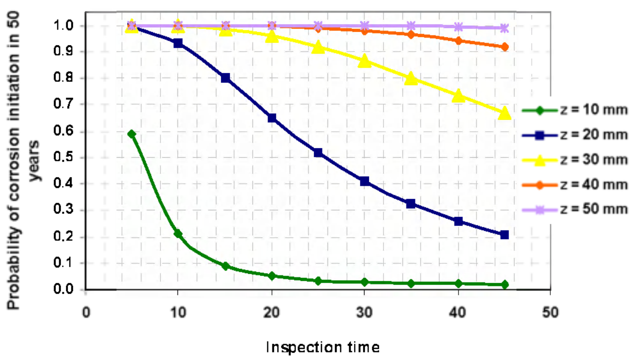

Figure 6. Probability of corrosion initiation at depth of reinforcement (t=50 years) given an indication of corrosion initiation at depth z (carbonation model).

Examining the probabilities of corrosion initiation for a negative indication, Malioka and Faber (2003) and Wicki (2003), these have been much smaller in the carbonation model compared to the ones in the chloride model something that can again be explained by that carbonation processes slowly; thus a negative indication of corrosion initiation at a certain depth makes it relatively not viable to have corrosion at the depth of reinforcement within the referred period. Nevertheless comparisons of this kind would be of more use if the models were corresponding to the same structure but data are still required in order to carry out such a research.

The quality of the inspection under examination may be expressed through the likelihood i.e. the probability of indication of corrosion initiation given that corrosion initiates at some specified point in time (Figures 7 and 8) and of the probability of indication of corrosion given that corrosion will not actually initiate before the referred period, see Malioka and Faber (2003) and Wicki (2003). In mathematical terms the likelihood is given by the conditional probabilities in the numerator of Equations (13) and (14). It can be seen for example that the likelihood of a current passing through the system in the chloride model is low for an inspection time of 5 years particularly at depths greater than 30 mm. For the case of carbonation induced corrosion an inspection in 5 years is rather unlikely to provide an indication of corrosion initiation. Therefore it will not be advantageous carrying out this type of test at an early stage. What is noticed in general is a shift to the left of the likelihoods in the case of corrosion due to carbonation that is another sign of the slow progress of carbonation relatively to the progress of chloride induced corrosion. It is quite improbable that a current passes through the ring configuration at depths more than 25 mm even for an inspection time of 45 years; while in the chloride model, for the same inspection time, the likelihood of a current passing through a ladder rung positioned in 60 mm has a value of about 70%.

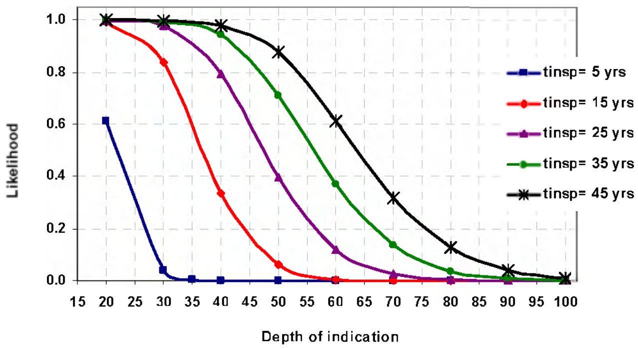

Figure 7. Likelihood of current passing at depth z given corrosion initiates at t=50 years (chloride model).

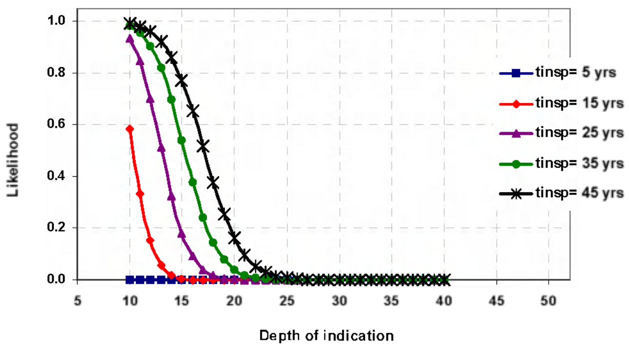

Figure 8. Likelihood of current passing at depth z given corrosion initiates at t=50 years (carbonation model).

For the case of no corrosion initiation in 50 years the likelihood profiles are similar but have significantly lower values particularly for the case of carbonation, Malioka and Faber (2003) and Wicki (2003).

Cover depth survey

General

Cover measurements are normally carried out on new structures to check that adequate cover has been provided to the steel according to the design specifications, Broomfield (1997). However this is not always the case and cover surveys normally also form part of principle inspections as they may help explain why a structure is corroding and show which areas are most susceptible to corrosion due to low cover. Moreover it is one of the most significant measurements since it can be used to interpret results from other NDE methods such as chloride analysis and carbonation depth measurements, TRRL (1980) as well as to update the probability of corrosion initiation as a function of time, Sterritt et al (2001).

The concrete cover thickness indicator

For the concrete cover measurement the following limit state has been used

$$M = d - d_{obs} + \varepsilon_d \tag{15}$$

where d_{obs} is the cover depth measured at the inspection and ε_d is the error associated with the measurement, here modeled as a normal distributed random variable with zero mean and standard deviation of 5 mm, BA 35/90 (1990). The updated probability of corrosion initiation given a cover thickness measurement may then be expressed as

$$P\left(T_I - t \le 0 \middle| d - d_{obs} + \varepsilon_d = 0\right) = \frac{P\left(d - d_{obs} + \varepsilon_d = 0 \middle| T_I - t \le 0\right) \cdot P\left(T_I - t \le 0\right)}{P\left(d - d_{obs} + \varepsilon_d = 0\right)} \tag{16}$$

In the same manner the posterior probability of visual corrosion may be expressed based on the limit state function of Equation (12). In Figures 9 and 10 the probability of corrosion initiation given a concrete cover measurement are shown for corrosion due to chloride ingress and carbonation respectively.

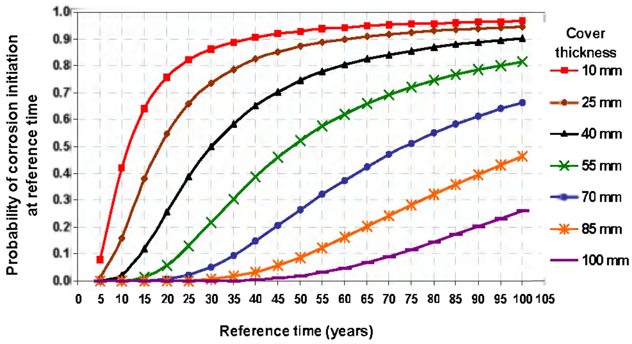

Figure 9. Probability of corrosion initiation given a cover thickness measurement (chloride model).

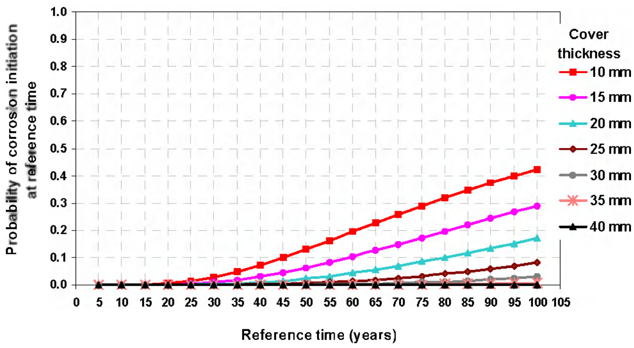

Figure 10. Probability of corrosion initiation given a cover thickness measurement (carbonation model).

It can be seen that the measurement of the concrete cover depth early in the structure's life can be very beneficial to predict the development of corrosion in the future. And to that extend the time of future measurements of a different nature may be planned to avoid major expenditures. If for example a cover measurement is undertaken say in 10 years after construction and a high value of cover is indicated then the expected probability of corrosion initiation is quite low, even negligible for corrosion due to carbonation. Thus other forms of testing may be planned for a much later point in time than the case would be if a small cover has been observed. In the latter case the structure would require frequent control and repair at a very early age thus leading to high costs. Moreover from the strength of the cover thickness indicator it is vastly

clear that carbonation does not progresses too fast and therefore a test such as carbonation depth measurements (as described later) could provide valuable information if carried out after some years from construction.
The likelihood of a specific observation, given by the conditional probability in the numerator of Equation (16), is shown in Figures 11 and 12. For a reference time of 5 years for example where corrosion initiates it can be seen that values of concrete cover greater than 30 mm are rather unlikely.

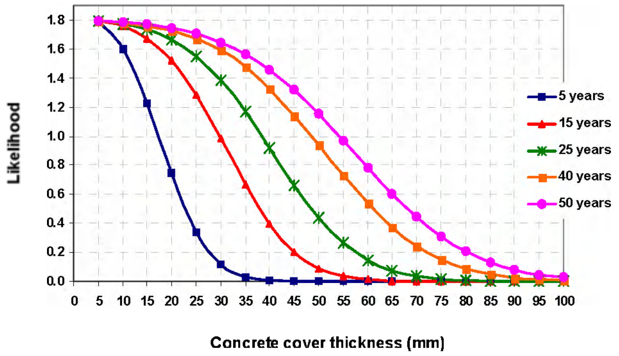

Figure 11. Likelihood of observed cover thickness given corrosion initiation (chloride model).

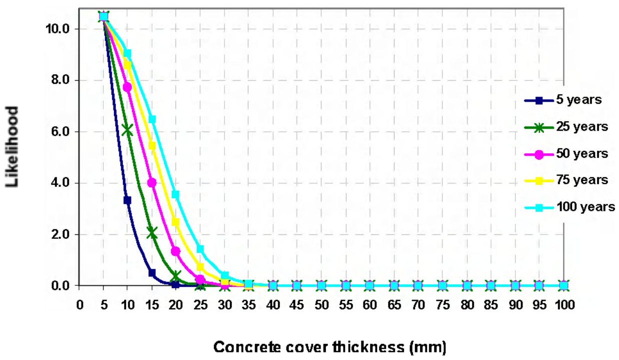

Figure 12. Likelihood of observed cover thickness given corrosion initiation (carbonation model).

Therefore based on initial knowledge about the time of corrosion initiation, if during an inspection a rather unlikely value of the concrete cover is observed then this could be interpreted as an erroneous measurement and should in any case give rise to an extra check.

Carbonation Depth Measurements

General

Carbonation of surface zone concrete will be accompanied by a loss of alkalinity of the concrete affected. Knowledge of the depth of carbonation may be useful when assessing potential durability and the likelihood of corrosion of the reinforcement. Carbonation depth measurements if compared with the average reinforcement cover may provide an estimation of the progression of depassivation with time. The most common method used to address carbonation depths is a solution of phenolphthalein sprayed on to a freshly exposed cut and or broken surface of the concrete. A detailed description of the technique may be found in Broomfield (1997) and Bentur (1997). There are some limitations to the method as described in Parrott (1987) and Broomfield (1997). However for most practical considerations the method is very accurate and reliable. Moreover is easy and fast to carry out. Phenolphthalein detects the change of pH by changing from colorless at low pH (carbonated zone) to pink at high pH values (uncarbonated zone).

The phenolphthalein indicator

The indicator is formulated based on the depth of carbonation measured with the solution of phenolphthalein. The time to corrosion initiation is given by Equation 9 where z is equal to $x_{obs} + \varepsilon$ where x_{obs} is the carbonation depth measured and ε is a factor corresponding to the uncertainty of the measurement and the actual depth of the carbonated zone, Wicki (2003). The probability of corrosion initiation at some point in the structure's life is given by Equation 13 and illustrated in Figure 13.

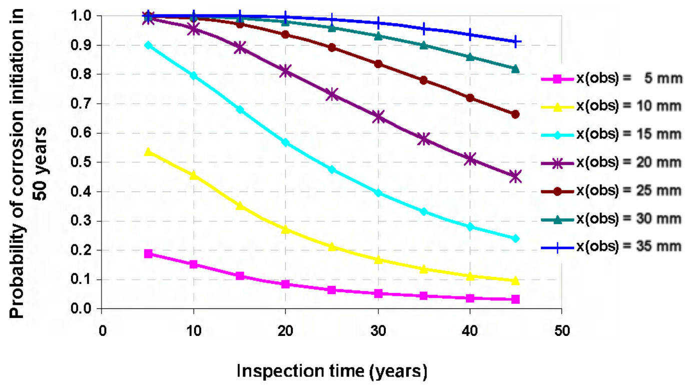

Figure 13. Probability of corrosion initiation (t=50 years) given a carbonation depth measurement.

For an inspection after 5 years of construction an observation of 5 mm indicates a small probability of corrosion initiation in 50 years. For larger carbonation depths the probability of corrosion initiation is high for early inspection times but decrease quite rapidly for inspections carried out later. Thus while an observed carbonation depth of 15 mm after 5 years of construction may form an alarm indication with a probability of corrosion initiation equal to 90%, the same observation after 20 years indicates a probability of something less than 60%. Moreover in the interior of the concrete (observations > 20 mm) a statement may be made after about 20 years. Following also a comparison with the probabilities provided by the ring system (Figure 6), it is evident that the last ones have a lower value for the same depth and inspection time. That is probably due to the fact that the carbonated zone is larger than the observed carbonation depth (see also Parrott (1987)).

The likelihood of a certain observation is shown in Figure 14. It is evident that an inspection 5 years after construction using phenolphthalein practically gives no indication of carbonation while if carried out after 15 years it is likely to obtain a small carbonation (~3 mm). However very large measurements are still not to be expected even after 45 years.

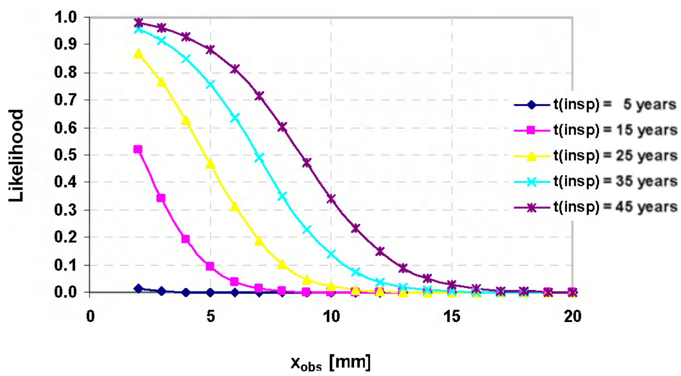

Figure 14. Likelihood of carbonation depth measurement given corrosion initiates at t=50 years.

Discussion and Conclusions

The quality and the relevance of inspection methods for condition assessment of concrete structures subject to corrosion have been considered using the concept of condition indicators in a Bayesian probabilistic framework. Following an approach previously utilized for the quantification of the quality and relevance for half-cell potential measurements, three non destructive evaluation methods have been examined namely macrocell technique, cover thickness measurements and carbonation depth measurements using a solution of phenolphthalein.

The rebar ladder and ring configurations of the macrocell technique provide a direct indication of corrosion initiation at different depths. It is a rather advantageous technique since it is installed once into the concrete and may provide long term monitoring of the progress of corrosion. Based on the formulation of the relevant indicator, i.e. the current passing through a ladder rung, it was seen that measurements at an early point in time after construction do not provide significant information about the future degradation states unless measurements are taken at rather low depths. Therefore if such a system has not been built into the concrete during construction the information gathered through the formulation of condition indicators may enable decisions upon an optimum time for installing the system and additionally upon the depth of installation and spacing of the ladder rungs (when the ladder configuration is being used). Moreover it was observed that there is not much information provided by indications at large depths (in the cases examined greater than 60 mm for the case of chloride ingress and greater than 40 mm for the case of carbonation) for inspection times up to 30 years. However the state of the next level of the system in depth has not been utilized within the present study. In normal situations, i.e. where the ingress of chlorides is not very advanced this approximation is not expected to be significant; however, for situations where the prior probability of initiation of corrosion at the system where no current is running is high the approximation could lead to errors. This aspect should be considered in more detail in the future.

The studies indicate that the cover thickness measurements are more beneficial if carried out at an early point in time. Based on the observation and the results for the strength of the indication the time of a critical degradation state may be predicted and interventions may be planned accordingly in order to avoid major future expenditures. The use of cover meter measurements as a condition indicator is proposed for quality control of newly built structures and if carried out early may be useful in the planning of other inspection methods. In the present paper the condition indicators have been examined individually. Nevertheless the combined use of different indicators containing supplementary information may provide a more consistent interpretation of inspection results also in relation to more than one state of interest such as durability and strength. For example it has been observed, Vu and Stewart (2000), that a structure with a high water/cement ratio and relatively good concrete cover presents a reduced compressive strength if compared to a structure with lower water/cement ratio and lower concrete cover. The cover thickness and the water/cement ratio utilized as indicators thus provide different information for different states of interest.

For indications by phenolphthalein the formulation of the relevant indicator has shown that a low observed carbonation depth does not provide much information regarding the progress of carbonation. The same applies for large carbonation depths observed up to 20 years. Moreover it was seen that it is rather unlike to get an indication of carbonation very early in the structure's life. That is because carbonation is a quite slow process and that should be taken into account when planning this type of test. Furthermore in order to predict with better accuracy the development in time of the carbonation process the concrete cover value should be taken into account.

In conclusion the formulation of condition indicators seems very promising in inspection and maintenance planning of concrete structures providing a tool for a consistent quantification of the information contained in inspection results. More work should be directed to a thorough mapping of the quality of different inspection methods together with their relevance for the different deterioration processes prevailing throughout the service lives of different classes of structures under different environmental conditions.

Acknowledgements

The Swiss National Science Foundation is greatly acknowledged for their financial support to the present work as part of the project "Indicators for the Durability and Service Life of Concrete Structures" with number: 2100-066707.

References

BA Advice (1990). *BA Advice Note 35/90* Department of Transport, Highways, Safety and Traffic. Inspection and Repair of Concrete Highway Structures. Crown, UK.

Benjamin, J.R. and Cornell, C.A. (1970). *Probability, Statistics and Decision for Civil Engineers*. McGraw-Hill, New York.

Bentur, A., Diamond, S. and Berke, N.S. (1997). *Steel Corrosion in Concrete*. E & FN Spon. London, UK.

Broomfield, J.P., (1997). *Corrosion of Steel in Concrete*. E&FN Spon. Great Britain.

Cusson, D. (2002). *Monitoring Performance of Corrosion Inhibitors in a Reinforced Concrete Bridge*. APWA International Public Works Congress, NRCC/CPWA Seminar Series "Innovations in Urban Infrastructure". Canada.

DuraCrete (1999). *Probabilistic Performance based Durability Design of Concrete Structures*. BRITE EURAM Project no. 1347.

Faber, M.H. (1997). *Risk Based Structural Maintenance Planning*. Probabilistic Methods for Structural Design, Solid Mechanics and its Applications Vol. 56: 377-402.

Faber, M.H. and Gehlen, C. (2002). *Probabilistischer Ansatz zur Beurteilung der Dauerhaftigkeit von bestehenden Stahlbetonbauten*. Journal of Beton- und Stahlbetonbau, 97. Heft 8: 421-429. ISSN 0005-9900, Ernst & Sohn.

Faber, M.H. and Sorensen, J.D. (2000). *Indicators for Assessment and Inspection Planning*. IBK Bericht Nr. 266, June 2001. Birkhaeuser Verlag Basel.

Faber, M.H. and Sorensen, J.D. (2002). *Indicators for Inspection and Maintenance Planning of Concrete Structures*. Structural Safety Vol. 4 (No 120): 377-396

JCSS (2001). *Probabilistic Assessment of Existing Structures*. A publication of the Joint Committee on Structural Safety. RILEM publications, France.

Lentz, A. (2001). *Potentialmessungen zur Unterhaltsplannung bei Stahlbetonbauwerken*. Diploma Thesis, Swiss Institute of Technology Zurich (ETH).

Madsen, H.O. et al (1986). *Methods of Structural Safety*. Prentice-Hall, Inc., New Jersey.

Malioka, V. and Faber, M.H. (2003). *Condition Indicators for Inspection and Maintenance Planning*. Accepted by ICASP9 (9th International Conference on Applications of Statistics and Probability in Civil Engineering), San Francisco, California, USA, July 6th-9th 2003.

Parrott L. (1987). *A review of Carbonation in Reinforced Concrete*, British Cement Association.

Raupach M. and Schiessl P. (2001). *Macrocell sensor systems for monitoring of the corrosion risk of the reinforcement in concrete structures*, NDT&E International, Vol. 34, 2001.

Schiessl, P. and Raupach, M. (1992). *Monitoring System for the Corrosion Risk of Steel in Concrete Structures*. Concrete International Vol. 14, Nr. 7: 52-55.

Sterritt, G., M.K. Chryssanthopoulos and Shetty, N.K. (2001). *Reliability Based Inspection Planning for RC Highway Bridges*. International Conference Malta, March 21-23 2001.

Torrent, R.J. (1992). *A Two-Chamber Vacuum Cell for Measuring the Coefficient of Permeability to Air of the Concrete Cover on Site*. Materials and Structures Vol . 25, Nr. 150: 358-365.

TRRL Laboratory Report 953 (1980). *A Survey of Site Tests for the Assessment of Corrosion in Reinforced Concrete*. Transport and Road Research Laboratory, England.

Vu, K.A.T. and Stewart, M.G. (2000). *Structural Reliability of Concrete Bridges Including Improved Chloride-Induced Corrosion Models*. Structural Safety Vol. 22: 313-333.

Wicki, R. (2003). *Zustandsindikatoren für die Inspektions-und Unterhaltsplanung von Betonbauten.* Diploma Thesis. Departement Bau, Umwelt und Geomatik. Institut für Baustatik und Konstruktion, Gruppe Risiko und Sicherheit, Swiss Institute of Technology (ETH), Zurich.

Observed Chloride Penetration in Marine Concrete Structure after 20 years in North Sea Environment

R.B. Polder[1], M.R. de Rooij[2], H. de Vries[3], J. Gulikers[3]
[1]TNO Building and Construction Research, Delft, The Netherlands
[2]Delft University of Technology, Faculty of Civil Engineering and Geosciences, Microlab, Delft, The Netherlands
[3]Ministry of Transport, Public Works and Water Management, Civil Engineering Division, Utrecht, The Netherlands

Abstract

On site and laboratory testing was performed with regard to durability of the Eastern Scheldt Storm Surge Barrier. The structure was built in the 1980s on the South West coast of The Netherlands. The Barrier stretches about 2800 m across the Eastern Scheldt estuary including two islands and consists of 68 openings, each with cast in-situ piers connected by precast beams and precast bridge elements and steel sliding doors. All concrete was made of blast furnace slag cement with a low w/c, using river sand and either dense river gravel or lightweight coarse aggregate. Cover depth to the reinforcement was measured. In the laboratory, chloride profiles, thin section microscopy and compressive strength and splitting tensile tests were performed. Chloride penetration is discussed with regard to concrete type and exposure. Service life calculations are presented using the DuraCrete model for chloride ingress induced corrosion initiation.

Introduction

The Eastern Scheldt Storm Surge Barrier is located on the south-western coast of the Netherlands. The barrier was designed in the 1970s to protect the low-lying land against flooding by sea water. The main structure was designed for a service life of 200 years using then existing service life prediction models (Gulikers, 2000); the bridge superstructure was designed for 50 years. The dominant deterioration mechanism was thought to be chloride ingress induced reinforcement corrosion. The barrier was built in the period 1980-1986 and a few chloride profiles were taken in 1989 and 1994. Presently, the owner is considering the need for maintenance of the bridge superstructure in 2005-2010.
Intensive inspection and sampling of four test areas on the barrier was carried out as part of a larger project for CUR research committee B82 'Durability of Marine Concrete Structures' (DuMaCon). The DuMaCon program aims to collect data and to validate the service life model developed in the EU sponsored program DuraCrete (DuraCrete R17, 2000). Additionally, large scale chloride profiling on the bridge superstructure was done for evaluating the need for maintenance. This paper concentrates on analysis of chloride ingress in the bridge superstructure.

DuraCrete model for chloride penetration induced reinforcement corrosion

The DuraCrete model for chloride penetration induced reinforcement corrosion assumes chloride transport by diffusion, based on a constant chloride surface content, a time-dependent diffusion coefficient and a threshold level for the critical chloride content at the reinforcement. All three variables depend on the exposure condition, which for marine structures is divided in submerged, tidal, splash and atmospheric zones. The corrosion process is schematised in two phases: an initiation phase in which chloride penetrates towards the reinforcement until the critical chloride content is reached; and a propagation phase in which actual corrosion takes place until an unacceptable level of damage is reached. Chloride penetration is the dominant process during the initiation phase. Figure 1 shows an example of chloride profiles at two ages and the relationship to the two phases.

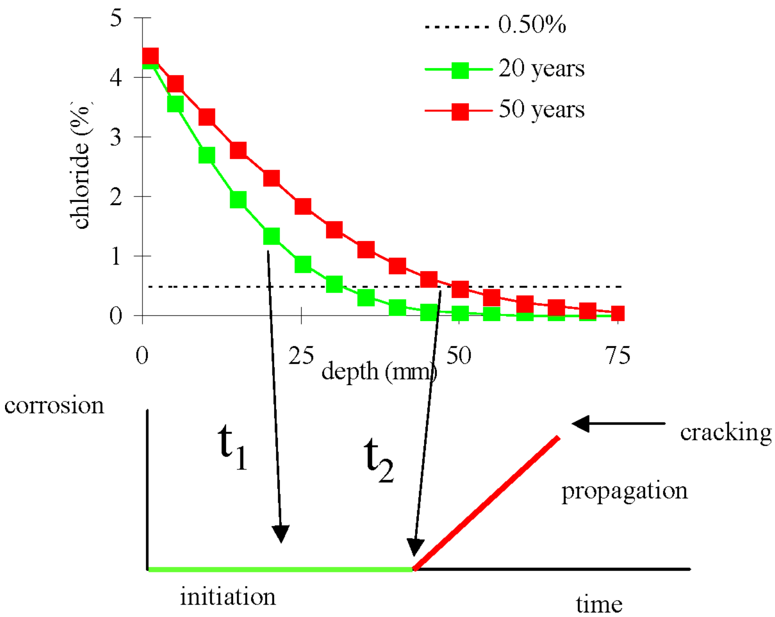

Figure 1. Chloride profiles in a concrete structure in marine environment at two ages and their relationship to the two phases in the corrosion process; assumed critical chloride content 0.50 % by mass of cement.

Since the 1970s, chloride transport in concrete has been modelled as a diffusion process using Fick's second law. For the case of penetration from the outside into chloride-free concrete and C_s and D_{app} are constants, the chloride content C(x,t) at depth x (for instance at the front of the rebars) and at time t is then given by:

$C(x,t) = C_s\,[1 - \mathrm{erf}\,(x/2\sqrt{(D_{app}\,t)}]$ (1)

with C_s *the chloride surface content* and D_{app} *the (apparent) chloride diffusion coefficient* and erf *the error function.*

From repeatedly investigated exposure sites and other work, it was discovered that the diffusion coefficient was not constant but appeared to decrease with time (Bamforth et al., 1994 and Maage et al., 1996), and more strongly so for blended cements (blast furnace slag and fly ash cements) than for Portland cement. This led to the introduction of time-dependency of the diffusion coefficient (Maage et al., 1996 and Thomas et al., 1999), which can be modelled by an exponential decrease:

$D_t = D_0\,(t_0/t)^n$ (2)

with D_t and D_0 being respectively, the diffusion coefficients *at the time of inspection* t and *at a reference time* t_0 (for instance from a test at early age in the laboratory); the *aging exponent* n takes values between 0 and 1, depending on the environment and the cement type. The DuraCrete model further includes a set of *conditional coefficients* expressing the influence of the environment, the cement type and the curing, here together denoted K. The chloride content C(x,t) at depth x and at time t is then given in the DuraCrete model by:

$C(x,t) = C_s\,[1 - \mathrm{erf}\,(x/2\sqrt{\{K\,D_0\,(t_0/t)^n\}})]$ (3).

A slightly different time dependent solution has been proposed by Visser et al. (2002) and a modified formulation of K was proposed by Gehlen (2000). DuraCrete provides a simple model for calculating C_s from the exposure zone and the concrete composition:

$C_s = A_{c,Cl}\,(w/b)$ (4)

with $A_{c,Cl}$ a *regression coefficient* depending on the cement type and the exposure zone; and (w/b) the *water-binder ratio.*

The DuraCrete Final Technical Report (2000) provides tabulated values for n and (individual coefficients together forming) K, based on exposure and field data and expert opinion.

Currently, in the design stage for a new structure, the DuraCrete approach can be applied as follows. The chloride surface content is calculated for the particular exposure zone. The intended cover depth is chosen and a limit value for the diffusion coefficient at 28 days is calculated using equation (3) from the desired length of the initiation period. Concrete compositions with potentially suitable diffusion coefficients are selected from a database. Then trial mixes are made and their diffusion coefficients are tested at 28 days age using a compliance test, for which DuraCrete has chosen the Rapid Chloride Migration (RCM) test (Tang, 1996). RCM is a short term test based on acceleration of chloride penetration by an electric field. If necessary, the composition is modified until satisfactory D_0 values are obtained. Several major structures have been designed for service life using this method. It should be noted that for new structures, D_0 is known (measured by accelerated laboratory testing) and C_s is predicted.
DuraCrete does provide both a full-probabilistic approach as well as partial (safety) factors for a Load and Resistance Factor Design (LRFD) based approach for calculating probabilities of failure. However, a probabilistic approach is outside the present scope. In the present paper, all considerations given pertain to *mean* times-to-corrosion.

Layout of the structure

The storm surge barrier across the Eastern Scheldt has a length of 2800 m and is located in the south-western part of the Netherlands at the estuary of the Eastern Scheldt bay into the North sea. The barrier consists of three parts - Hammen, Schaar and Roompot - named after the original tide-ways it now crosses (Figure 2). It contains 68 openings, of which 62 can be closed by steel sliding doors. During normal weather conditions the sliding doors are fixed in a raised position to allow free tidal movements. Only during periods of strongly increased flooding risk, the barrier will be closed by lowering the sliding doors.

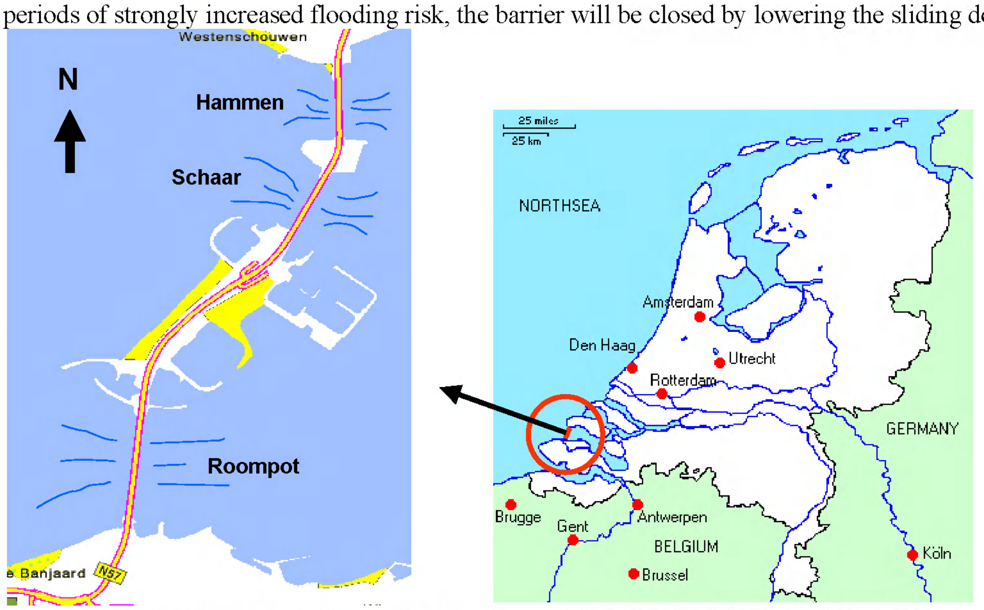

Figure 2. Location of Eastern Scheldt Storm Surge Barrier and its three parts: Hammen, Schaar, Roompot.

The complete barrier comprises of 65 concrete piers at 45 m centres. The piers are linked by threshold beams on the seabed and upper beams about 1 m above mean sea level (NAP). The piers, threshold beams and upper beams are manufactured from gravel concrete with reinforcing and prestressing steel. A motorway "bridge" made of prestressed concrete box girder elements is built over the storm surge barrier. A schematic is shown in Figure 3.
The bridge-elements consist of dense gravel concrete (max. grain size 32 mm, 325 kg blast furnace CEM III/B per m^3, w/c 0.44) except for one span at each abutment, made of lightweight concrete with expanded shale as coarse aggregate (max. 8 mm, 400 kg CEM III/B per m^3, w/c 0.38). Each section was precast and steam cured for 2 hours (max. temperature 37 °C). The bridge-elements were cast and placed mainly in 1984. The bridge-decks protect the sides of the box girders against direct precipitation. When the sliding doors are in the raised position, the lower part of the door is about NAP + 1 m and the highest point varies between NAP + 5.90 m and + 11.90 m, the highest doors being near the midpoint of the original tide-ways.

This means that the sliding doors will be shielding (part of) the bridge-elements facing the North Sea, as western wind predominates. Furthermore, turbulent airstreams have been observed around the structure.

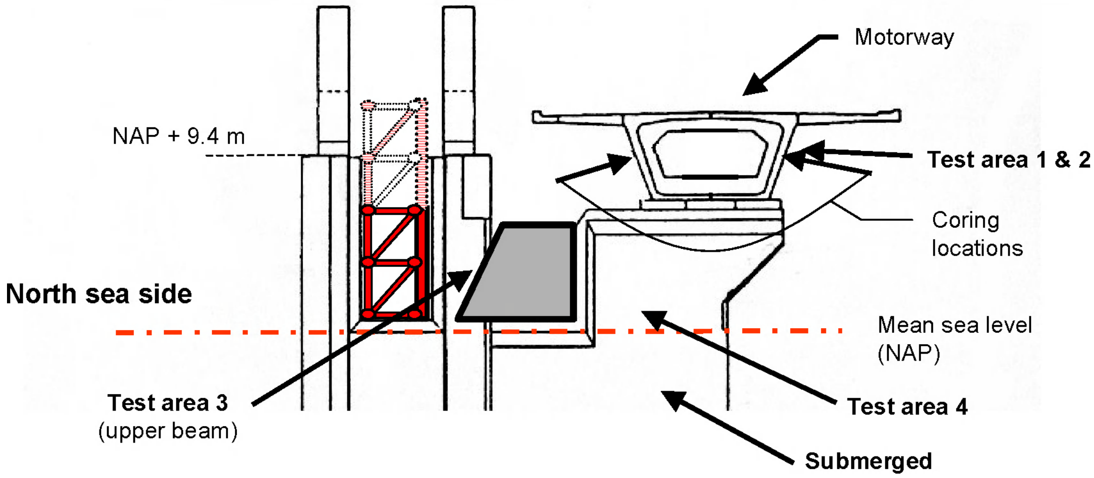

Figure 3. Cross section of a pier with the motorway bridge and test/core sampling locations and the difference between the smallest and largest sliding doors (in raised position).

Inspection and sampling

Intensive investigation for DuMaCon took place in 2002 on four test areas (1 - 4, each about 1 x 1 m^2, position indicated in Figure 3). In test area 1, six cores were taken from a gravel concrete bridge element; test area 2 was on lightweight concrete, both on the Eastern Scheldt side. Cores of 100 mm diameter were taken for compressive and tensile strength and RCM testing. For the additional profiling investigation (API), about 75 cores were taken from bridge elements, indicated “coring locations”, from both sides of the bridge and along the complete length of the barrier. Chloride profile cores had 50 mm diameter. They were diamond-sawn in slices which were subsequently powdered, dissolved in concentrated hot nitric acid and titrated according to Volhard. DuMaCon testing involved 6 slices (010, 10-20, .. 50-60 mm) and API testing involved 4 slices (for gravel concrete 10-18, 18-26, 26-34 and 34-42 mm; for lightweight concrete10-20, 20-30, 30-40 and 40-50 mm). Cover depth measurements on site were performed using a Hilti Ferroscan FS10. Additional cores were taken for microscopy.

Results

Concrete microscopy of 13 samples showed that the concrete was properly mixed and compacted. The cement type was blast furnace (>65% slag) and the degree of hydration was moderate to high, the apparent w/c about 0.45 for gravel concrete. Paste-aggregate bond was good (gravel) to excellent (lightweight). No deterioration was observed due to frost, sulphate, alkali-silica reaction or leaching. Carbonation depths were generally a few mm. Deposition of products of reaction between cement paste and sea water (such as calcite, brucite, ettringite) were restricted to a layer of 0.1 mm thickness on the surface and inside voids down to a few mm depth. The overall quality was good (gravel) to excellent (lightweight). Compressive (tensile) strength for both mixes was c. 60 MPa (4 MPa), wet density about 2420 (gravel) and 1830 (lightweight) kg/m^3. The electrical resistivities measured on samples stored under water or vacuum saturated showed high values, typical for dense slag cement (about 600 Ωm). Steel in gravel concrete in test area 1 had a mean cover depth of 41.1 mm (SD 1.4 mm). The outer bars in lightweight concrete in test area 2 had a mean cover depth of 47.7 mm (SD 3.2 mm).
The results of the chloride profiles allow discussion for various combinations of factors that could influence chloride penetration: concrete composition (lightweight/normal weight); North Sea/Eastern Scheldt side; height of sliding door. Some factors appear to interact. On the North Sea side chloride ingress is much lower for lightweight concrete than for gravel concrete. This is also the case on the Eastern Scheldt side, but the difference is small compared to the scatter (Figure 4). For gravel concrete the chloride penetration

is higher on the North Sea than on the Eastern Scheldt side, while for lightweight concrete it is higher on the Eastern Scheldt side than on the North Sea side.

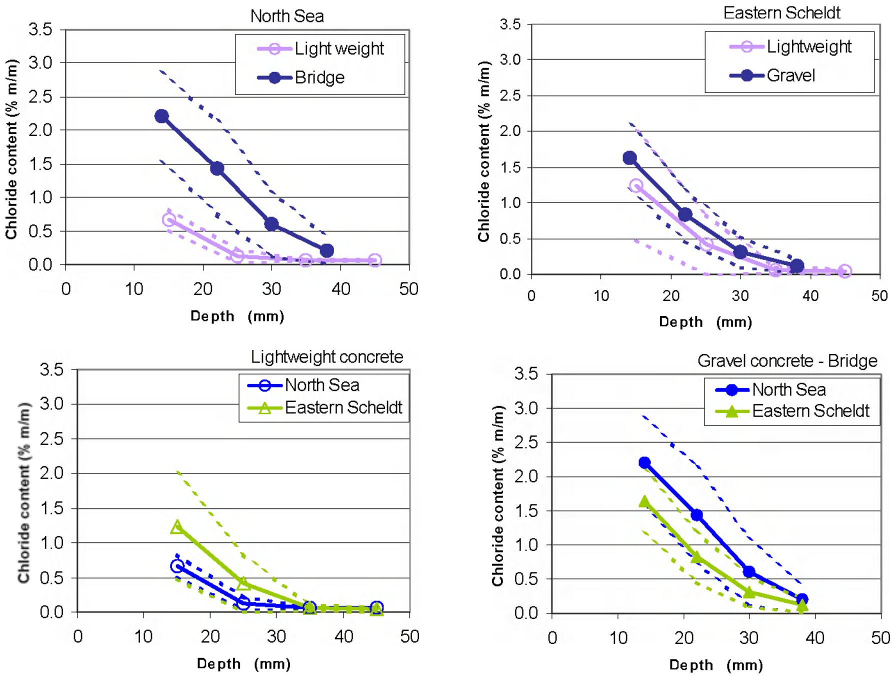

Figure 4. Various groups of chloride profiles; solid lines: mean; hatched lines: mean plus or minus one standard deviation.

Another source of variation is related to the position along each of the three parts of the barrier (Hammen, Schaar, Roompot), associated with the height of the sliding doors. Mean chloride profiles have been plotted per bridge element and compared to the relative door height in Figure 5. It shows that the lowest chloride ingress has occurred behind the highest doors and that the effect is most pronounced in the Schaar part of the barrier. It appears that the sliding doors shield and partly protect the bridge-elements at the North Sea side. On the Eastern Scheldt side no noticeable effect of the size of the doors was found. This issue concerns only gravel concrete, as there are no sliding doors in front of the lightweight spans.
Chloride penetration profiles are analysed and characterised by C_s and D_{app} obtained by fitting the diffusion equation (1). Table 1 summarizes the data. The chloride surface content C_s for gravel concrete appears to be rather constant at 4.5 to 5%, except for the influence of slide door height (extremes 6.8 and 2.6%). C_s for lightweight concrete is lower on the North Sea side, but apparently it varies between locations on the Eastern Scheldt side. The mean value of 9 cores taken along the complete barrier was 5.5%, with extremes 3.1% and 10.8%. In test area 2 (Hammen 16), the mean of 6 cores taken within 1 m^2 of concrete surface was 3.1%. D_{app} is remarkably constant for gravel concrete, about 0.30 * 10^{-12} m^2/s; and lower but slightly more variable for lightweight concrete between 0.13 and 0.21 * 10^{-12} m^2/s.

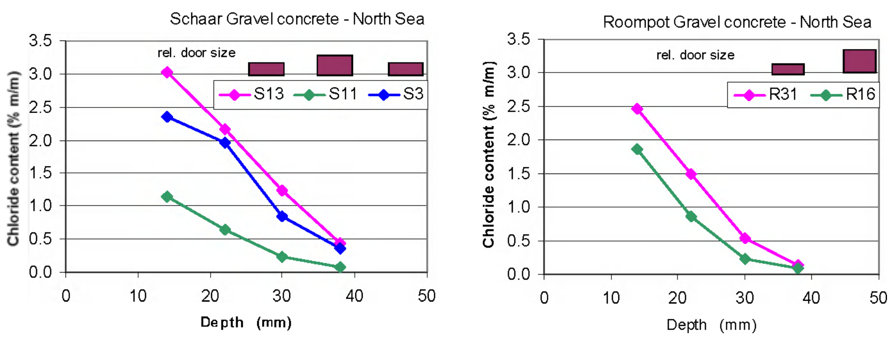

Figure 5. Chloride profiles in gravel concrete on North sea side, on different positions along the flow channels Schaar and Roompot with relative door height indicated

Table 1. Mean values (and standard deviation) for Cs and Dapp for various combinations of exposure, concrete composition and door height; API additional multiple core testing; TA1, 2 Test Areas 1 and 2; Schaar 13 and 11 are sub-sets of API; number of cores per data-set.

Concrete	Gravel			lightweight			gravel	
Dataset (door height)	API	API	TA 1	API	API	TA 2	Schaar 13 (5.9 m)	Schaar 11 (11.9 m)
Exposure side	North Sea	Eastern Scheldt	Eastern Scheldt	North Sea	Eastern Scheldt	Eastern Scheldt	North Sea	North Sea
Surface content (% by mass of cement)	4.9 (1.3)	4.4 (1.3)	5.3 (1.5)	3.4 (1.2)	5.5 (2.4)	3.1 (0.1)	6.8	2.6
Diffusion coefficient ($x10^{-12}$ m^2/s)	0.33 (0.12)	0.26 (0.09)	0.28 (0.12)	0.13 (0.05)	0.21 (0.09)	0.13 (0.01)	0.27	0.30
no. of cores	14	12	6	16	9	6	1	2

Verification of the DuraCrete model for existing structures

In principle, the DuraCrete model is also applicable to existing structures. In most cases, of course some assumptions on parameters are then necessary, for instance the 28-day diffusion coefficient is not available because it has not been measured at the time of construction (the test did not exist yet). Generally, a set of chloride penetration profiles taken during an inspection at time t_{insp} will be the starting point. The profiles are analysed using equation (1), which produces a set of pairs of C_s and D_{app} values. So for an existing structure having been inspected once, C_s is known and D_0 is not. One way of further analysis is to apply the tabulated aging exponent and environmental coefficient from DuraCrete (calculating backwards in time), to arrive at D_0. Having obtained D_0 this way, predictions for future penetration can now be made similarly to the situation for new structures by applying equation (3).
In the DuraCrete model C_s is generated using equation (4). Using C_s from the measured profiles instead of calculating it from equation (4), C_s can be used for updating the service life design. This implies that the aging exponent and the conditional coefficients are reliable. At present, however, their reliability seems limited. In this study, C_s and D_{app} are taken from the inspection profiles together with D_0 values from a database for similar concrete compositions; D_t (from calculation) is supposed equal to D_{app} (from inspection). Then the age exponent n and the conditional coefficient K that shows the best fit to all sets of C_s, D_{app} and D_0 are compared to the tabulated values.

Table 2. Input and output of DuraCrete model calculations for 3 cases and time-independent calculation from observed values for surface chloride content and chloride diffusion coefficient; gravel concrete bridge element Hammen 8; RCM at 28 days estimated 4.5 * 10-12 m2/s; critical chloride content 0.5%, cover 41.1 mm

test area	1			
location	gravel concrete bridge element Hammen 8, Eastern Scheldt side			
Case	1	2	3	4
DuraCrete zone	atmospheric	splash	splash + C_s (update)	observed
n (-)	0.85	0.60	0.60	D constant
C_s (% mass of cement)	1.4	3.0	5.3	5.3
D_t (10^{-12} m^2/s)	0.18	0.28	0.28	0.28
t_{ini} (year)	$9*10^7$	230	92	34

Table 2 reports the input and output of DuraCrete model calculations for gravel concrete from test area 1 for 3 cases. A 4^{th} case is added, based on the observed mean values for the chloride surface content and diffusion coefficient and taking these values constant over time. The chloride profiles as calculated by the model are compared to the obtained mean profile in test area 1 and presented in Figure 6. Case 1 is based on equation (3) and the conditional coefficients for the atmospheric zone. Both the chloride surface content and diffusion coefficient are strongly underestimated by the DuraCrete model. Time-to-initiation is about 90 million years. This value is obviously too optimistic; it is not realistic to design the bridge elements for atmospheric exposure. Case 2 is based on equation (3) and splash zone exposure coefficients. The time-to-initiation is about 230 years. Because the calculation underestimates C_s, this result must be regarded as

optimistic; the diffusion coefficient is correctly predicted. Case 3 is based on splash zone exposure with the surface content taken (updated) from the measurements (Table 1). Time-to-initiation then becomes about 90 years; this seems to be a realistic mean time-to-initiation. For the worst case position with the highest surface content due to limited shielding by a low sliding door (Schaar 13, Table 1), a time-to-initiation was found of 65 years. Case 4 is based on the observed surface content and a constant diffusion coefficient, without considering its decrease over time. This approach does not take into account the beneficial effect of further hydration or the effect of drying out (n = 0) and is outside the DuraCrete model. Time-to-initiation for Case 4 is about 34 years. Summarising, the DuraCrete prediction (using the splash zone model) for the gravel concrete bridge elements is too optimistic because the chloride surface content is underestimated. Updating the calculations by using the observed surface content after 18 years appears to produce a realistic *mean* time-to-initiation.
It appears that whatever model is used, the *mean* time-to-initiation of the gravel concrete bridge element is well beyond the required design life of 50 years. The lowest time-to-initiation may be found for elements where the steel sliding doors are lowest.

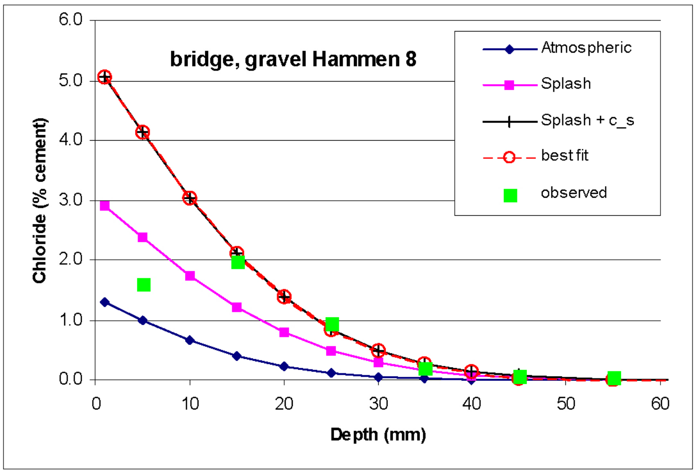

Figure 6. Chloride profiles calculated from DuraCrete for case 1, 2 and 3 (updated Cs) and the mean profile as observed on test area 1 (case 4); gravel concrete

Table 3. Input and output of DuraCrete model calculations for 3 cases and for observed values for surface chloride content and (constant) chloride diffusion coefficient; lightweight concrete bridge element Hammen 16; RCM at 28 days assumed 4.5 * 10-12 m2/s [8]; critical chloride content 0.5%, cover 47.7 mm

test area	2			
location	lightweight concrete bridge element Hammen 16, Eastern Scheldt side			
case	1	2	3	4
DuraCrete zone	atmospheric	splash	splash + n (update)	observed
n (-)	0.85	0.60	0.74	D constant
C_s (% mass of cement)	1.4	3.0	3.0	3.0
D_t (10^{-12} m^2/s)	0.18	0.28	0.13	0.13
t_{ini} (year)	$7*10^8$	485	$8*10^4$ *750 mean for API*	160 *66 mean for API*

Table 3 reports the input and output for 3 cases of DuraCrete model calculations for lightweight concrete (test area 2). As above, a 4th case is added based on the observed values for surface chloride content and (a constant) diffusion coefficient. Case 1 is based on DuraCrete calculations for the atmospheric zone. The time-to-initiation is 700 million years. This value is clearly too optimistic, due to the strongly underestimated chloride surface content. Case 2 is based on splash zone exposure. The time-to-initiation is about 500 years. Because the calculation overestimates D_t, strictly spoken this result must be regarded as pessimistic. Case 3 is based on splash zone exposure, however, the age exponent is increased (updated) in order to arrive at the diffusion coefficient found after 18 years. The time-to-initiation is 80,000 years. Case 4 is based on the observed surface content and diffusion coefficient. As mentioned above, this is outside the DuraCrete model. The time-to-initiation for Case 4 is about 160 years.

Taking into account the data of all cores from lightweight concrete (see also Table 1), the results change slightly. The cores taken for API from the Eastern Scheldt side show a much higher surface content and a much higher diffusion coefficient than test area 2. Consequently, the time-to-initiation is significantly shorter. By coincidence, test area 2 is the most favourable location of all lightweight elements sampled. Using the mean values for all lightweight concrete on the Eastern Scheldt side (Table 1), a modified DuraCrete splash zone calculation produced a mean time-to-initiation of 750 years (instead of 80,000, case 3) and using a constant diffusion coefficient produced a mean time-to-initiation of 66 years (case 4), see Table 3. Taking the results from Schaar 17, where chloride penetration was most advanced of all lightweight elements, the case 3 result was 275 years.
It appears that whatever model is used, the *mean* time-to-initiation of the lightweight concrete elements is well beyond the required design life of 50 years; it is probably beyond the required service life of the complete barrier (200 years). It should be noted, however, that the DuraCrete models may not apply for lightweight concrete, as the Final Technical report does not explicitly give parameter values for concrete made with lightweight aggregate.

Conclusions

A large scale investigation of chloride penetration was carried out on the Eastern Scheldt Storm Surge barrier, constructed on the North Sea coast in the 1980s. This paper concentrates on the bridge superstructure, located about 10 m above mean sea level.
Data after c. 18 years marine exposure show advanced chloride penetration, which depends on concrete type (gravel or lightweight), orientation (North Sea or Eastern Scheldt side) and shielding by sliding doors. Analysis of the measured chloride profiles using Ficks second law of diffusion shows relatively consistent values for chloride surface content and diffusion coefficient for gravel concrete; however, more scatter was present in these parameters for lightweight concrete.
Further analysis was made using the DuraCrete time-dependent diffusion model. Modelling as atmospheric exposure strongly underestimated chloride penetration. Reasonable fits to observed profiles were obtained using coefficients for splash zone exposure. Mean times-to-corrosion-initiation calculations resulted of about 230 years (gravel concrete) and about 500 years (lightweight concrete). When the prediction is updated using the observed chloride surface content from the inspection (gravel concrete) or the age exponent (lightweight concrete), updated mean times-to-corrosion-initiation calculations resulted between 65 and 90 years (worst case and mean for gravel concrete) and between 275 and 750 years (worst case and mean for lightweight concrete).
Regarding the DuraCrete model and its tabulated coefficients, chloride transport in the gravel concrete was modelled correctly, but the surface chloride content was underestimated. For lightweight concrete, the surface content was modelled correctly but the rate of chloride transport was overestimated.

References

Gulikers, J. (2000). *Service Life management of a concrete structure exposed to marine environment.* Life Prediction and Aging Management of Concrete Structures. RILEM proceedings PRO 16, Cannes, 113-121.

DuraCrete R17 (2000). *DuraCrete Final Technical Report*, The European Union – Brite EuRam III, DuraCrete – Probabilistic Performance based Durability Design of Concrete Structures, Document BE95-1347/R17, May 2000; CUR, Gouda, The Netherlands.

Bamforth, P.B., Chapman-Andrews, J. (1994). *Long term performance of RC elements under UK coastal conditions*, Proc. Int. Conf. on Corrosion and Corrosion Protection of Steel in Concrete, ed. R.N. Swamy, Sheffield Academic Press, 24-29 July, 139-156

Maage, M., Helland, S., Poulsen, E., Vennesland, O., Carlsen, J.E. (1996). *Service life prediction of existing concrete structures exposed to marine environment*, ACI Materials Journal, 602-608

Thomas, M.D.A., Bamforth, P.B. (1999), *Modelling chloride diffusion in concrete – Effect of fly ash and slag*, Cement and Concrete Research, Vol. 29, 487-495

Visser, J., Rooij, M.R. de, Gaal, G. (2002). *Time dependency of chloride diffusion coefficients in concrete*, RILEM TMC Workshop Madrid.

Gehlen, C. (2000). *Probabilistische Lebensdauerbemessung von Stahlbetonbauwerken*, Deutscher Ausschuss für Stahlbeton 510**,** Berlin

Tang, L. (1996). *Electrically accelerated methods for determining chloride diffusivity in concrete*, Magazine of Concrete Research, Vol. 48, 173-179

Deterioration and maintenance of concrete bridges based on spatial variability of structural properties

Ying Li[1], Ton Vrouwenvelder[1], Geert Henk Wijnants[2]

[1]Delft University of Technology, Stevinweg 1, P.O. Box 5048, 2600GA Delft, The Netherlands,
[2]Netherlands Organization for Applied Scientific Research (TNO)
[1]e-mail: y.li@ct.tudelft.nl

Abstract

It is observed by simulations in this paper, that with considering spatial variability of structural properties in the model, it is more realistic and reasonable to do the reliability analysis of structural deterioration and therefore improve the decision making of optimal maintenance strategy for the structures.

Introduction

Due to variety of physical and chemical processes, concrete bridges as well as other civil engineering structures deteriorate and may reach, after some time, a minimum acceptable level of performance. At that point in time maintenance (including inspection, repair and replacement) has to be carried out correctly in order to avoid that the structure has insufficient reliability or efficiency. As many short-cyclic maintenance processes have shown, a more preactive approach can be much more cost-effective. For practice it is important that costs involved in those maintenance activities can be predicted and where possible be optimized. As most parameters in those processes are random, probabilistic-based reliability analysis is utilized, that in general the probability of failure (e.g. spalling of concrete due to corrosion) is calculated as a function of time. However, the fact that a lot of parameters show spatial random variability, which are intricately linked with dependencies on temperature, w/c ratio, cement type, humidity and workmanship (Chryssanthopoulos 2002), is not included in most of the studies. Until now, work concerning with spatial variation for durability aspects of concrete structures have: Hergenroeder & Rackwitz 1992, Engelund 1998, 1999, and 2000, Sϕrensen 1999, Sterritt 1999, Karimi & Ramachandran 1999, Chryssanthopoulos 2002, Vu and Stewart 2001, 2002 and Faber 2002.
If there is no spatial variability, the result at one point is applicable for the whole element, or the complete structure or even the total set of structures. This is of course not very realistic. This paper is concerned with the deterioration analysis caused by chloride initiated corrosion of concrete bridges with due regards to the effect of spatial variability of two random variables by means of Monte-Carlo method. Under the condition of various spatial correlations, the probability of having at least one corrosion spot in the beam is calculated as well as the probability distribution for the corroded area during the lifetime. The method of considering spatial variability in the reliability calculation and its impact on the inspection or repair decision are discussed. Furthermore, different repair strategies and their costs are compared with consideration of the effect of spatial variability.

Impact of spatial variability on concrete degradation

Schematisation of modeling concrete bridge

A bridge can be divided into regions based on different criteria, for example based on vulnerability to damage, environmental condition, structure, loading or construction situation, etc. In this case a region is defined as "an area that, based upon efficiency considerations for repair actions, should be treated as a whole". Each region is looked as an independent spatial random field. For simplicity, the research focuses on only one rectangular region from a concrete bridge. It is assumed to have a length of 18m and width of 6m (Figure 1) and divided into *n* equal elements along the x direction and *m* equal elements along the y direction. The deterioration of concrete is assumed to be caused onlyhby chloride-induced rebar corrosion. The designed lifetime is set to be 80 years.

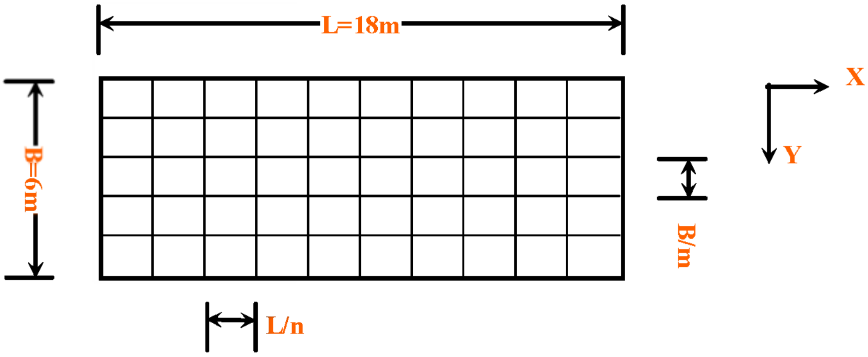

Figure 1. Modelling one region of the structure

Modeling of deterioration

The chloride ingress at the rebar in year *t* can be modelled by the following equation:

$$c_{Cl}(t)=c_{Cl;s}\cdot erfc\frac{a}{\sqrt{4\cdot D_{Cl}\cdot t}} \tag{1}$$

where $c_{Cl}(t)$ concentration of chloride at time t in depth a [%]
$c_{Cl;s}$ concentration of chlorides at the concrete surface [%]
a concrete cover [mm]
t time [year]
D_{Cl} achieved chloride diffusion coefficient [m^2/year]

For each element in Fig.1, the corresponding limit state function is given by:

$$Z_i = c_{Cl;cr,i} - c_{Cl,i}(t) \tag{2}$$

Where $c_{Cl;cr,i}$ is the critical value of concentration of chloride for *i* element [%]
We assume here, for simplicity, that an element is failed when corrosion of rebar begins. In mathematical terms it means that $Z_i < 0$ (equation (2)). The total corroded area is calculated by summing up the area of all failed elements in each year. From a practical point of view, this represents the total number of "work packages" to be handled.

Modeling for spatial variability of physical properties

We assume the region is a homogeneous Gaussian field. Normally, of course, random fields are seldom homogeneous. Many properties may differ systematically form point to point. From many applications, however, the homogeneous Gaussian field may be a helpful tool in the statistical description of spatial random properties.
Here the variables c_{Cls} (concentration of chlorides at the concrete surface) and *a* (concrete cover) in equation (1) are assumed to show spatial random fluctuation. A homogeneous Gaussian field is assumed to describe the random spatial properties. The correlation between different points ($\rho_{\Delta x}$) for each variable can be calculated by equation (3), which is only a function of distance between two points (Vrijling 1999, Vrouwenvelder 2002):

$$\rho(\Delta x) = \rho_0 + (1-\rho_0)\exp\left(-\left(\frac{\Delta x}{d}\right)^2\right) \quad (3)$$

where: ρ_0 parameter indicates a common source of correlation for all elements

Δx distance between two points (sections) [m]

d fluctuation scale [m]

The other two variables in equation (1), the critical value of chloride concentration ($c_{Cl,cr}$) and the chloride diffusion coefficient (D_{Cl}), are assumed to be either fully correlated (ρ_0=1) or independent for each element (Table 1).

Input data

Overview of input properties of variables is shown in Table 1, where 3 typical different correlation scenarios are considered.

Table 1 Three different correlation scenarios

				Scenarios		
				(a) small correlation	(b)median correlation	(c) full correlation
Variable	Distribution	Mean	Std. Dev.	$\rho_0 \mid d$	$\rho_0 \mid d$	$\rho_0 \mid d$
C_{Cls}	Normal	1.7%	0.425%	0 | 2	0.5 | 2	1 | ∞
$C_{Cl;cr}$	Normal	0.5%	0.1%	independent	1 | ∞	1 | ∞
D_{Cl}	Normal	2E-5 m^2/Year	7E-6 m^2/Year	independent	1 | ∞	1 | ∞
a	Normal	60 mm	9.7 mm	0 | 2	0.5 | 2	1 | ∞

Calculation

The Monte Carlo Method is carried out in the space with independent standard normal variables. Therefore, transformation is carried out to the correlated random variables (c_{Clsi} and a_i in this case) due to spatial variability (Thoft-Christensen 1987). We can find the relationship by Choleski's decomposition between transformation matrix and correlation coefficients ρ_{ij} obtained by equation (3).

Results

According to the scenarios (a)~(c) mentioned in the above section 'input data', the probabilities of having at least one failed element (called system failure) are obtained in Figure 2, and the proportions of corroded area are shown in Figure 3.

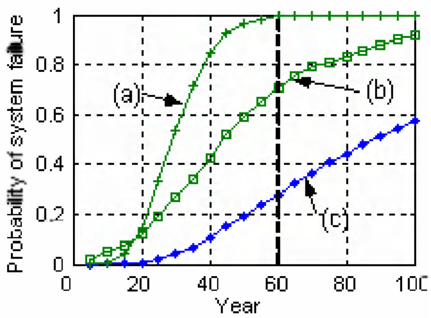

Figure 2. Probability of system failure (a) Small correlation; (b) Median correlation; (c) Full correlation

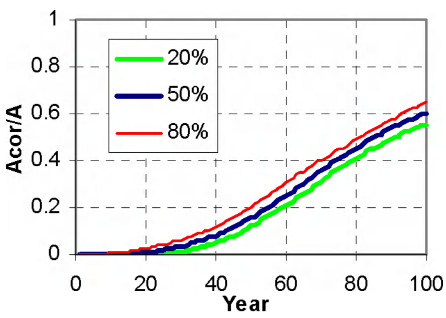

Figure 3a. small correlation

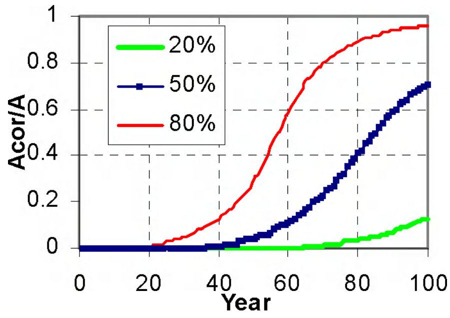

Figure 3b. median correlation

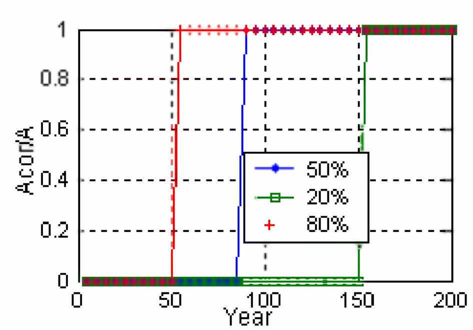

Figure 3c. full correlation

Figure 3a till c. Proportion of corroded area by chloride ingress with time under different correlations

(Acor is the corroded area in the beam [m^2] and A is the total area of the beam [m^2])

We can see a large difference between different correlations and Figure 3. The probability of system failure is becoming larger with the decrease of the correlations between elements. For example in year 60 in Figure 2, the system probability of failure will be changed from 25%, 70%, to 100% if element correlations changes from "full" to "median" and to "small".

In Figure 3, we can get the view of damage situation of the region in each year for different correlation properties.

- We use the value at year 60 again in Figure 3b) to explain the meaning of three curves in each figure of Figure 3:
  - The value of Acor/A is 0.1 for the curve of 50%, which means in year 60:

    $$P\left\{\frac{A_{cor}}{A} \leq 0.10\right\} = 50\%.$$

  - The value of Acor/A is 0.6 for the curve of 80%, which means in year 60:

    $$P\left\{\frac{A_{cor}}{A} \leq 0.6\right\} = 80\%.$$

  - The value of Acor/A is 0 for the curve of 20%, which means in year 60: $P\left\{\frac{A_{cor}}{A} = 0\right\} = 20\%$.

    That also means probability of no damage is 20%.
- In Figure 3, the difference between the value of curve 50% and 80% or curve 50% and 20% is becoming larger if the elements are changing from small correlation to full correlation.

- If the elements are fully correlated (ρ_0=1), only two possibilities: no corrosion or completely corroded. (Figure 3c). For example in year 60, it has 20% probability that the beam is completely corroded, and 80% probability that the beam is in acceptable condition.

Optimal maintenance strategy with spatial variability

The above results indicate the importance of taking into account spatial variability effects in our inspection and maintenance strategy. Still for the same region, we now study the optimal maintenance strategy under the condition that elements show median correlation (scenario b).

Repair criterion

Maintenance and repair are carried out in order to ensure that the structure is able to fulfil some design requirements throughout its lifetime with sufficient reliability. In the Netherlands like many other countries, repair of concrete structures is primarily based on visual inspection that recording the signs of deterioration, that is to say the criterion for onset of repair are usually based on a given percentage of the surface of the structure shows signs of corrosion such as rust stains, cracks, or delamination of concrete. Generally 1% is used. When relating this empirical repair criterion to the theoretical acceptance criterion, in this case a failure means the initiation of bar corrosion. The time, from initiation of corrosion until signs of corrosion are visible needs a period of time about 10-20 years. In order to make the link between the repair criterion "1% deteriorated by visual means", the corresponding proportion of corroded area is about 0.2 (20%) in this study, which is selected as the repair criterion. Of course, using feedback from practice, another criterion can be used as well.

Optimal maintenance strategy

The optimal maintenance strategy is the one that has the minimum expected maintenance cost, where sometimes under the reliability constrains (Frangopol 1997, 1999, Engelund 1998, 1999, Enright 1999, Estes 1999, Gasser 1999):

Min E[C], While sometimes, $P_f \le P_f^{\max}$ (4)

where:

E[C] is the total expected maintenance cost and $P_f^{\max}$ is the required upper limit for some critical event.

For strategies in this paper, the principle is taken that repair begins at a time before the probability of life failure, because signs of deterioration represent no immediate threat to the load-bearing capacity of the structure. Therefore, we will determine the optimal maintenance strategy simply only by minimizing the total expected cost E[C] (Fujimoto 1992, Estes 2001).

$$E[C]=E[C_{insp}]+E[C_{rep}] \tag{5}$$

$$E[C_{insp}] = \sum_{k=1}^{mi} \frac{C_{insp,k}}{(1+r)^{t_k}} \tag{6}$$

$$E[C_{rep}] = \sum_{j=1}^{nr} \frac{C_{rep,j}}{(1+r)^{t_j}} \tag{7}$$

$$C_{rep;j} = C_0 + C_u \cdot \sum A_{rep;i} \tag{8}$$

where:	C_{insp}	inspection costs	mi	inspection times
	t_k	inspection year	r	interest rate
	C_{rep}	the repair costs	nr	repair times
	t_j	repair year	C_0	Initial repair cost
	C_u	Unit repair cost	$\sum A_{rep;i}$	Total repaired area

Repair strategies

In this report, a repair strategy is defined as a set of predefined and related measures that have been coupled in order to reach the goals set in an effective way. After repair criterion is reached, four repair strategies (based on the visual inspection every year) are compared:

- **Strategy 1** (Figure 4a)**:**
  Repair will be carried out only for the corroded area.
- **Strategy 2** (Figure 4b)**:**
  Repair will be carried out for the corroded area and its horizontal (main direction) adjacent area.
- **Strategy 3** (Figure 4c)**:**
  Repair will be carried out for the corroded area and its surrounding area.
- **Strategy 4:**
  After 15% of the surface shows signs of corrosion (repair criterion is 20%), do maintenance coating for the region. For calculation, we assume 5mm more for the concrete cover after each coating.

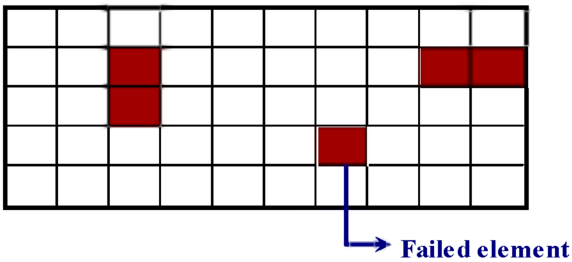

Figure 4a. Strategy 1: Only failed elements will be repaired

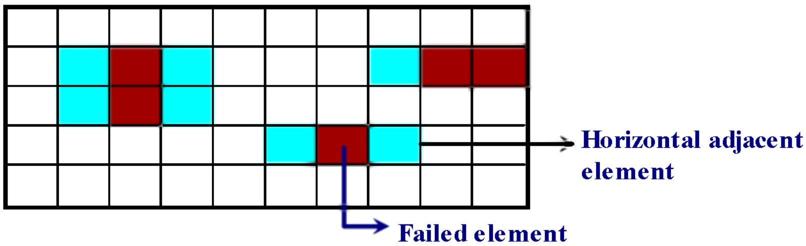

Figure 4b. Strategy 2: Besides failed elements, its horizontal (main direction) adjacent elements will be repaired

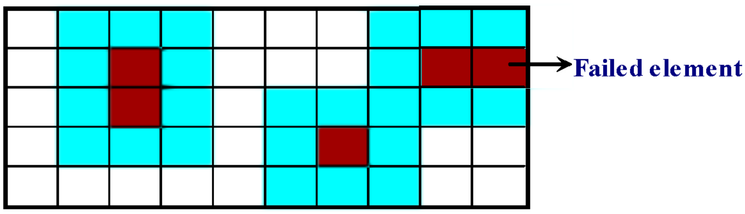

Figure 4c. Strategy 3: Besides failed elements, all its surrounding adjacent elements will be repaired

Cost input

1. Repair cost
   Initial repair cost (C_0): 5000 Euro
   Unit repair cost (C_{u1}): 2000 Euro/m
2. Visual inspection cost: 50 Euro/time
3. Maintenance cost (Coating): 500 Euro/m^2

Results

Effects of different strategies are shown in Figure 5. A performance indicator ***I*** (Figure 6) is introduced to compare the effects among different strategies that is given by I=A1/A2. The indicator presents a relative value in order to provide comparison for the situation with maintenance to the situation without maintenance. The smaller I is, the more impact the strategy has on the reduction of the amount of corroded area. It is clear Strategy 3 provides the best performance.

Expected yearly repair costs for different strategies are shown in Figure 7. The total expected costs are compared in Figure 8. Strategy 2 has the minimum total cost. Figure 9 shows that Strategy 2 performs slightly less than strategy 3 but yet for 40% of the costs involved, it is the optimal strategy for this case.

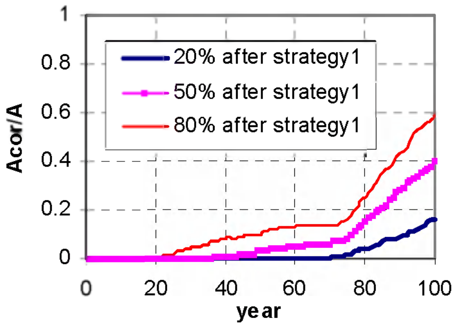

Figure 5a Effect of strategy 1

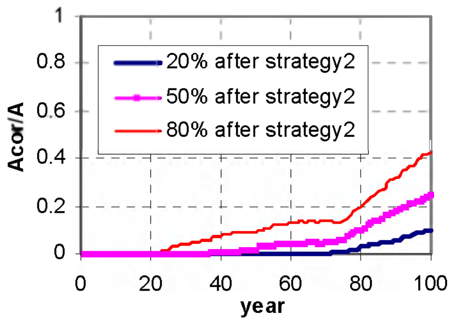

Figure 5b. Effect of strategy 2

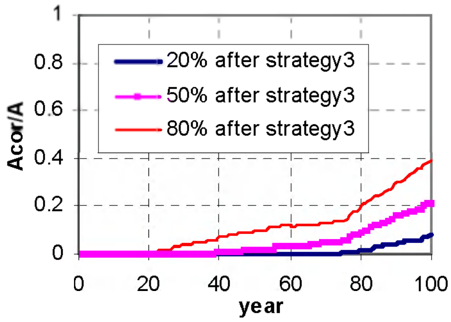

Figure 5c. Effect of strategy 1

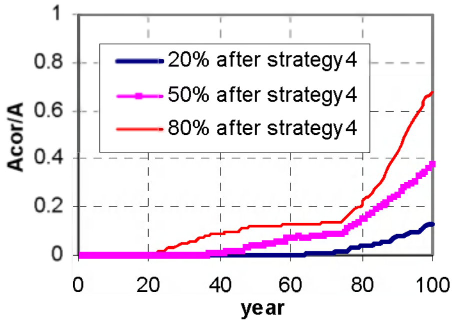

Fig.ure 5d. Effect of strategy 2

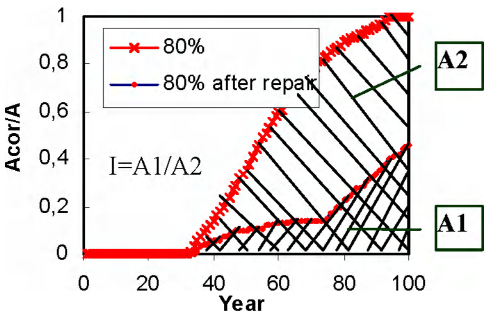

Figure 6a. Calculation of performance indicator *I*

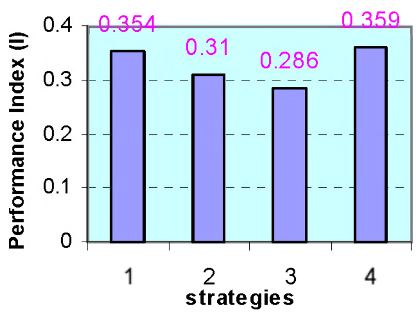

Figure 6b. Performance indicator of strategies

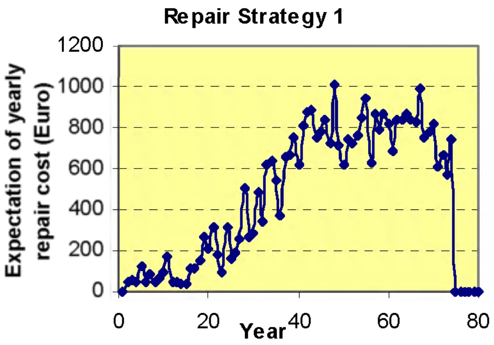

Figure 7a. Expected yearly repair cost of strategy 1

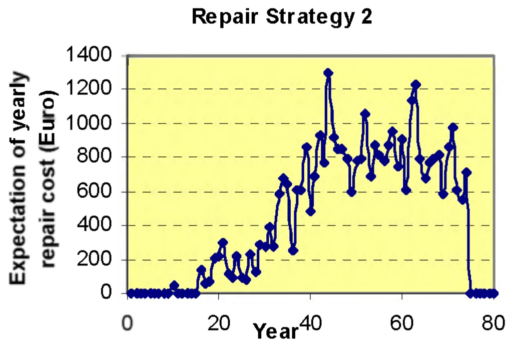

Figure 7b. Expected yearly repair cost of strategy 2

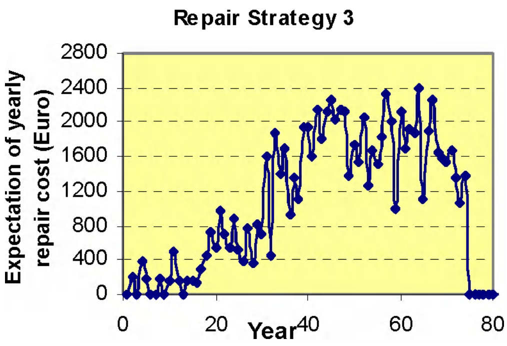

Figure 7c. Expected yearly repair costs of strategy 3

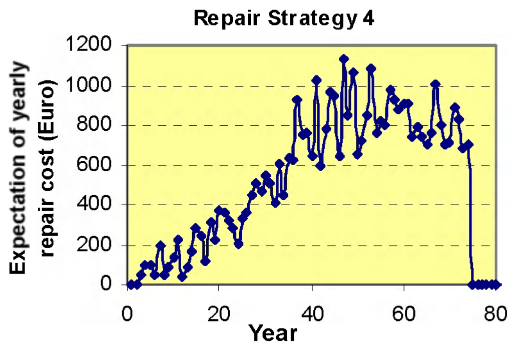

Figure 7d. Expected yearly repair costs of strategy 4

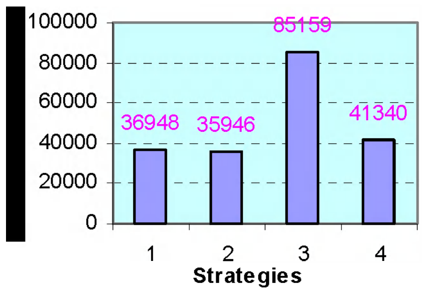

Figure 8. Total expected costs of strategies

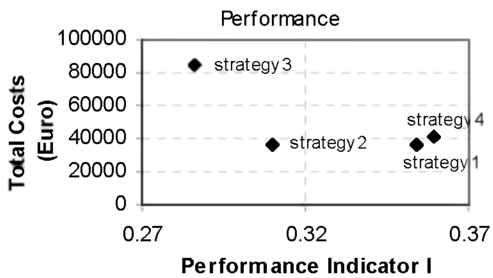

Figure 9. Performance Index versus Costs

Conclusions

Most present models for the reliability analysis of durability aspects neglect random variation in space. From the analysis in this paper, we know it is not possible to deal properly with the real situation.

In this paper it has been proved that the degree of spatial variability of random variables has a great impact on the final results of the structural reliability analysis. The probability of failure and proportion of corroded area have been calculated based on the spatial variability of physical properties of structure. The results are used to do the deterioration prediction and assessment of optimal repair strategy.

It is important to get a good impression of the spatial parameters such as fluctuation scale d and auto-correlation coefficient ρ_0, in order to find out a realistic and useful description of the condition of the structural elements.

The results presented in this paper remain theoretical, real data are under collection for properly selecting the model parameters in the next work.

Acknowledgement

The work is finished under the supervision of Prof. Ir. A.C.W.M. Vrouwenvelder who is my promotor. At the same time, I am sincerely grateful to Ir. G. Wijnants (TNO), my supervisor; Prof.dr.ir. J.C. Walraven, my co-promoter; Dr. ir. C. van der Veen, my supervisor. Besides, I would like to thank all the people who have been involved in my project, especially P.H.A.J.M.van Gelder for his constructive comments during my research work, A. van Beek, A.M. Bots, M.H. Klatter, J.Bakker, M.H. Djorai for sharing their experience in practical and Q.Chen for his kind help on my theory study and computer programming. Finally the financial support of the Ministry of Transport, Public Works and Water Management, Netherlands School for Advanced Studies in Construction as well as the Netherlands Organization for Applied Scientific Research is greatly appreciated.

Reference

Duracrete, (1998). *Modelling of Degradation*. Probabilistic performance based durability design of concrete structures, Document BE95-1347/R4-5, the European Union- Brite EuRamIII, Dec. 1998.

Branco, F.A. and de Brito, J. (1996). *Bridge management from design to maintenance*. Recent advances in bridge engineering, J.R. Casas, F.W. Klaiber and A.R. Mari (eds.), Barcelona, p.76-98.

Cheung, M.S. and Kyle, B.R. (1996). *Service life prediction of concrete structures by reliability analysis*. Construction and Building Materials, 10(1), p.45-55.

Chryssanthopoulos, M. K and Sterrit, G. (2002). *Integration of Deterioration Modelling and Reliability Assessment for Reinforced Concrete Structures*. ASRANet International Colloquium 2002.

CUR (1997), *CUR 190:Kansen in de civiele techniek*. Directoraat-Generaal Rijkswaterstaat, (in Dutch).

Engelund, S. (2000). *Planning of Inspections of Concrete Structures*, Proceedings of the ESRA workshop on Reliability and risk-based inspection planning, Dec. 2000.

Engelund, S., and Sorensen, J.D. (1998). *A Probabilistic Model for Chloride Ingress and Initiation of Corrosion in Reinforced Concrete Structures*. Structural Safety, Vol. 20, p.69-89.

Engelund, S., Sorensen, J.D., and Sorensen, B. (1999). *Evaluation of Repair and Maintenance Strategies for Concrete Coastal Bridges on a Probabilistic Basis*. ACI Materials Journal /March-April 1999.

Enright, M.P. and Frangopol, D.M. (1999). *Condition prediction of deteriorating concrete bridges using Bayesian updating*, ASCE, Journal of Structural Engineering: 1118.

Enright, M.P. and Frangopol, D.M. (1999). *Maintenance planning for deteriorating concrete bridges*. Jornal of Structural Engineering, Vol 125, No.12.

Enright, M.P., and Frangopol, D.M. (2000). *Survey and Evaluation of Damaged Concrete Bridges*. Journal of Bridge Engineering: 31.

Estes, A.C. and Frangopol, D.M. (1999). *Repair optimisation of highway bridges using system reliability approach*. Journal Structure Engineering, 125(7), p.766-775.

Estes, A.C., and Frangopol, D.M. (2001). *Minimum Expected Cost-oriented Optimum Maintenance Planning for Deteriorating Structures: Application to Concrete Bridge Decks*. Reliability Engineering and System Safety 73, p.281-291.

Estes, A.C., and Frangopol, D.M. (1998). *Optimum Reliability-based Inspection/Repair Strategy based on Minimum Expected Cost*. Structural Safety and Reliability, Shiraishi, Shinozuka and Wen (eds.), Balkema, Rotterdam, (ISBN 90 54109785).

Faber, M.H., and Sφrensen, J.D. (2002). *Indicators for inspection and maintenance planning of concrete structures*. Structural Safety 24, p.377-396.

Frangopol, D.M. (1998). *A framework for balancing reliability and life-cycle cost in bridge management.* Reliability and optimisation of structural systems, Nowak, A.S. and Szerszen, M.M. (eds.), USA, p.3-13.

Frangopol, D.M. (2000). *Optimal management of structures based on balancing reliability and life-cycle cost.* Proceedings of Workshop on Optimal Maintenance of Structures, Delft, p.51-69.

Frangopol, D.M., and Estes, A.C. (1999). *Optimum Lifetime Planning of Bridge Inspection and Repair Programs*, Structural Engineering International, 3/99.

Frangopol, D.M., Lin, K., and Estes, A.C. (1997). *Life-cycle cost design of deteriorating structures.* Journal Structure Engineering, 123(10), p.1390-1401.

Fujimoto, Y. and Swilem, A. M. (1992). *Inspection Strategy for Deteriorating Structures Based on Sequential Cost Minimization Method.* 1992 OMAE, Volume II, Safety and Reliability, ASME 1992.

Gasser, M. and Schuëller, G.I. (1999). *On optimising maintenance schedules by reliability based optimisation.* Case studies in optimal design and maintenance planning of civil infrastructures systems, Frangopol, D.M. (ed.), p.102-114.

Hergenroeder, M., and Rackwitz, R. (1992). *Zur statistischen Instandhaltungsplanung fuer bestehende Betonbauwerke. (Plannning the structural maintenance of existing concrete buildings).* Baouingenieur. Vol. 67, no.11, p.491-492, 493-496,497.

JCSS, Joint Committee on Structural Safety (2000), *Assessment of Existing Structures*.

Karimi, A.R. and Ramachandran, K. (1999). *Probabilistic estimation of corrosion in bridges due to chlorination.* Application of statistics and probability. Melchers and Stewart (eds.), 2000, Rotterdam, ISBN 90 5809 086 8.

Papoulis A. (1965). *Probability, Random, Vairables and Stochastic Processes*. McGRAW-Hill Kogakusha, LTD.

Sφrensen, J.D. and Engelund, S. (1999). *Reliability-based optimal planning of maintenance of concrete structures.* Case studies in optimal design and maintenance planning of civil infrastructures systems, Frangopol, D.M. (ed.), p.224-235.

Sterritt, G., and Chryssanthopoulos, M. K. (1999). *Probabilistic limit state modelling of deteriorating RC bridges using a spatial approach.* Current and future trends in bridge design, construction and maintenance, Thomas Telford, p.518-528.

Stewart, M.G. (2001). *Reliability-based assessment of ageing bridges using risk ranking and life cycle cost decision analysis*. Reliability Engineering and System Safety, 74, p.263-273.

Thoft-Christensen P. and Murotsu Y. (1986). *Application of Structural Systems Reliability Theory.* Arpinger-Verlag.

Vrijling, J.K. (1999). *Probabilistic Design in Hydraulic Engineering*. Lecture notes for CTWA5310 (f30), Subfaculty of Civil Engineering, temporary issue.

Vrouwenvelder A.C.W.M. and Schiessl, P. (1999). *Durability Aspects of Probabilistic ULS Design.* Heron, 44(1), p.19-29.

Vrouwenvelder, A.C.W.M. and Vrijling, J.K.(2002). *Probabilitish ontwerpen.* Lecture notes b3, Subfaculty of Civil Engineering.

Vu, K. A.T. and Stewart, M.G.(2001). *Cracking and spalling reliability analysis considering spatial variability for reinforced concrete structures.* Structural Safety and Reliability, Corotis et al. (eds.), ISBN 90 5809 197

Vu, K. A.T. and Stewart, M.G. (2002). *Spatial variability of structural deterioration and service life prediction of reinforced concrete bridges*. First Int. Conf. On Bridge Maintenance, Safety and Management, IABMAS 2002, Barcelona, 14-17 July 2002.