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Planning of Inspection and Repair for Ship Operation

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ABSTRACT

Based on evaluation of fatigue failure probabilities for different service periods of marine structures, adaptive inspection and subsequent repair strategies are developed. It is explained how planning of inspection time intervals can be optimized with respect to total expected cost. The term adaptive shall underline that decisions taken after inspection are based on failure probabilities that include inspection results, i.e. the time interval to any subsequent inspection depends on the actual state of the considered structural element. The Paris–Erdogan crack propagation law is used with crack instability as the failure governing fatigue criterion. Stresses are determined as Gaussian random processes, which are based on linear stress calculations using advanced hydrodynamic wave load theory and the finite element method. Load and resistance parameters are assumed to be uncertain. The results of the finite element analyses and the reliability calculations are linked through a response surface program using Hermite polynomials. The method is exemplified with the determination of the optimal time interval to the first inspection of a hatch corner of a container ship operating on either a North Atlantic or a Pacific shipping route.

INTRODUCTION

Ships and offshore structures are subject to substantial fatigue caused by nearly permanent action of waves and other environmental and operational load cycles. Traditionally, such structures are inspected at regular time intervals and, possibly, repaired or even put out of service if the damage has reached critical values. Such inspections are costly and repair can be difficult, but high standards of quality must be maintained throughout all phases of fabrication, installation, and operation to protect men and environment from serious harm or damage. Thus, inspections and repair should be done as effectively as possible with due regard to costs involved, i.e. modern inspection and repair strategies should aim at highest possible degree of safety at lowest possible cost.

Detecting and measuring cracks is difficult and may not be very informative at short time intervals after a ship or offshore structure is put into service because normally not much damage has been accumulated. If late inspections are accomplished, however, serious damage may have been developed. Thus, inspection planning is a problem of optimizing inspection time intervals, amount of inspections, and thresholds for decisions whether to repair or not.

The problem so far appears to have mainly empirical solutions. Besides a basic study [Yang &Trapp 1974] very few references are available [Madsen 1987, Sørensen & Thoft–Christensen 1988]. Our present approach, which, in its basic concepts, was already outlined in [Fujita et al. 1989], is an adaptive inspection strategy based on cost optimization, now coupled via response surfaces [Schall et al. 1991] with modern hydrodynamic and structural analysis methods.

For the latter we make full use of the latest developments in numerical modeling of marine structures in their natural environment, both in theory and practical applications. This is possible because numerical techniques could be improved substantially, e.g. by determining hydrodynamic forces on a ship making direct use of 3–D linear potential theory of moving bodies in gravity waves [Östergaard et al. 1979, Papanikolaou et al. 1990]. Stresses and stress concentrations in structural members of a ship due to those forces are calculated by efficient finite element codes [Bathe et al. 1982]. Simultaneously, modern concepts and tools to model the random nature of most of the input parameters and the techniques to quantify structural failure probability in terms of probabilistic measures are available. Computer programs efficiently perform the required probability integrations for a large set of probabilistic models and arbitrary failure criteria for large dimensions [Hohenbichler al. 1987]. Recently developed programs handle complex computational tasks to calculate time variant failure probabilities [Bryla et al. 1990].

Response surfaces were introduced because a direct combination of state–of–the–art hydrodynamic and structural analysis with modern reliability computations is still not feasible. The numerical effort of a sound reliability analysis grows roughly quadratically with the number of basic uncertain variables. Interesting studies of the kind were, however, performed (we refer, for example, to [Mansour 1974, Ferro & Cervetto 1984, Akita 1988]), but simplifications either in the mechanical model, in the reliability model or in both models were made.

FATIGUE FAILURE PROBABILITY

We shall restrict our considerations to the crack propagation phase only but mention in passing that similar considerations can also be made for the endurance limit or initiation phase of crack development. Crack instability is taken as the governing failure mode. The types of steel and the environmental conditions of marine structures normally are such that the concepts of non–linear fracture mechanics apply. One of the simplest failure criteria is

$$(1) \qquad F(t) = \left\{ \frac{S_r}{\left[\frac{8}{\pi^2} \ln\left[\frac{1}{\cos(\pi S_r/2)}\right]\right]^{1/2}} - K_r(t) \leq 0 \right\}$$

where S_r and $K_r(t)$ are non–dimensional plastic collapse and fracture parameters, respectively. These quantities are defined by

$$(2) \qquad S_r \approx 2 \frac{S_p}{(S_y+S_u)}$$

$$(3) \qquad K_r(t) = \frac{K_I(t)}{K_{IC}} = \frac{Y(A(t))\sqrt{A(t)\pi}\ S_{ps}}{K_{I\,c}}$$

with S_p the peak stress at the net cross section of the structural element and S_{ps} the peak stress including macroscopic stress concentration and residual stresses. S_y and S_u are the yield strength and ultimate strength, respectively. K_{IC} is the fracture toughness, $A(t)$ the crack length at time t and $Y(A(t))$ a geometry factor. For a crack at the edge of a panel the geometry factor can be assumed constant Y_c=1.12. Other sometimes more realistic, experimentally based failure criteria can be found in [Milne et al. 1988].

For crack propagation a simple one–dimensional crack growth relationship is assumed

$$(4) \qquad \frac{dA(\tau)}{d\tau} = C\,(Y(A(\tau))\sqrt{A(\tau)\pi}\,\Delta S(\tau))^M$$

with $\Delta S(\tau)$ the stress range at $t=\tau$, C and M material parameters. This equation has to be integrated in order to determine the crack length A(t) at time t under the initial condition $A(0) = A_0$. The time dependent crack length A(t), which enters into (3), yields a time variant resistance threshold. For random loading an appropriate counting of the damage relevant stress cycles has to be performed. For simplicity of presentation it is assumed that the stress process is a zero mean Gaussian process with variation $\sigma_S^2 = m_0$ and regularity (band width) parameter $\alpha^2 = 1 - \epsilon^2 = m_2^2/(m_0 m_4)$, m_i being the i–th spectral moment of the process. The usually conservative peak counting method is applied for an effective stress range $\Delta S(\tau) = |2 S_{max}(\tau)|$ with $S_{max}(\tau)$ the magnitude of local maxima at time τ. Thus, the following analytical approximation for the crack length at time t can be obtained [Abdo et al. 1989]

$$(5) \qquad \psi(A(t)) \approx \psi(A_0) + t\,\frac{1}{2\pi}\sqrt{\frac{m_4}{m_2}}\,(2\sqrt{2m_0})^M \left[\frac{\epsilon^{M+2}}{2\sqrt{\pi}}\,\Gamma\!\left(\frac{1+M}{2}\right) + \alpha\Gamma\!\left(\frac{2+M}{2}\right)T\!\left(\frac{\alpha}{\epsilon}\sqrt{M+2}\right)\right]$$

$\psi(A(t))$ is a function obtained by integrating (4) with respect to $A(\tau)$. It depends on M and C in the Paris–Erdogan equation (4) and the particular form of the geometry function $Y(A(\tau))$. $T_\nu(.)$ is the central Student's t distribution with degree of freedom ν. The inverse function $\psi^{-1}(.)$ yields the crack length A(t) at time t which is needed in (1) and (3).

The spectral moments m_0, m_2, and m_4 are given in a quasi–analytical form using response surfaces as explained in [Schall et al. 1991].

Even under these simplifying assumptions no closed form solution for the time variant failure probability exists. If, however, interest is in small failure probabilities a well–known asymptotic formula, based on the assumption of Poissonian exits of the stress process out of the safe domain can be given [Leadbetter et al. 1983].

$$(6) \qquad P_f'(t|r,q) \sim 1 - \exp\left[-\int_0^t \nu^+(F(\tau|r,q))\,d\tau\right]$$

Herein, $\nu^+(.)$ is the outcrossing rate of the stress process $\{S(\tau)\}$ out of the safe domain into the time variant failure domain denoted by F(.) as defined in (1), r is a vector of time invariant random variables describing system properties and strength parameters (e.g. fracture toughness, yield and ultimate tensile strength, and initial crack length), and q is an ergodic sequence describing the long–term fluctuations of the environmental load process (e.g. sea state parameters as significant wave height H_v, wave period T_v, and main direction of seaway ϑ) and of the operational loads (e.g. ship speed V, and loading condition of the vessel). It is important that this failure probability is written as a conditional failure probability. Only then the aforementioned Poissonian nature of the failure events is ensured.

The outcrossing rate of the stress process $\{S(\tau)\}$ out of a safe domain as specified with (1) can be written somewhat more precisely in terms of the spectral moments m_i, the time variant equivalent resistance $R_E(\tau)$, and the still–water stresses μ_S

$$(7) \qquad \nu^+(F(\tau|r,q)) = \sqrt{\frac{m_2}{m_0}}\,\frac{1}{2\pi}\int_0^t \exp\left[-\frac{1}{2}\left[\frac{R_E(\tau)-\mu_S}{\sqrt{m_0}}\right]\right]d\tau$$

where the equivalent resistance is given implicitly in (1) but can be formulated explicitly as

$$(8) \qquad R_E(\tau) = \frac{(S_y+S_u)}{\pi}\,\mathrm{acos}\left[\left\{\exp - \frac{1}{8}\left[\frac{K_{I\,c}\ 2\pi}{Y(A(\tau))\sqrt{A(\tau)\pi}\ (S_y+S_u)}\right]^2\right\}\right]$$

The total failure probability can be obtained by taking the expectations with respect to **R** and **Q** in a certain manner. In [Schall et al. 1990] it was shown that the unconditional failure probability can be calculated according to the following asymptotic scheme with reasonable accuracy if the time t is sufficiently large compared to a characteristic fluctuation period of the sequence **Q**

$$(9) \qquad P_f'(t) \approx P(T < t) \sim 1 - E_{\mathbf{R}}\left[\exp\left\{-E_{\mathbf{Q}}\left[\int_0^t \nu^+(F(\tau|\mathbf{R},\mathbf{Q}))d\tau\right]\right\}\right]$$

where $E_{\mathbf{X}}[.]$ denotes expectation with respect to the random vector **X**, and T is the random time to failure. In general, the dimension of **R** and **Q** are far too large to perform the necessary integrations to obtain the expectations by a direct numerical integration scheme. However, it is possible to apply FORM/SORM concepts. Efficient numerical methods to carry out the calculation of the expectation with respect to parameters **R** and **Q** and the integral with respect to time τ on this basis can be found in [Bryla et al. 1990].

INSPECTION PLANNING

Consider first the simple case of a structural element with one planned inspection in between a pre-selected service time t_s. The time to inspection shall be defined and a decision shall be made whether to repair or not on a minimal cost basis. We assume that after repair the structural element has properties as if it was new. Further, inspection as well as repair time are assumed negligibly small compared to the time to inspection and the total service time.

If at time $t = t_I$ an inspection is performed, which, of course, is only possible if the structural element has survived up to this time, the actual state is observed. If no repair is required the posterior probability at $t = t_s - t_I$ is $P_f''(t_s - t_I)$. If this value or some damage indicator exceeds a given limit repair is required. The probability of repair is denoted $P_R(t_I)$. If the structural element is repaired the residual failure probability is denoted by $P_f'''(t_s - t_I)$. This failure probability is based on a new realization of the stochastic properties of the structural element. The situation is illustrated in Figure 1.

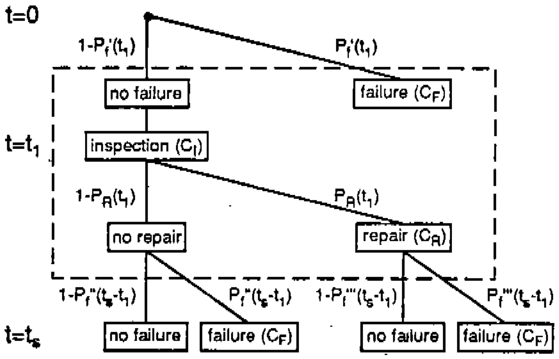

Figure 1: Decision Tree

Three types of cost are involved: the cost of inspection C_I, the cost of repair C_R and the cost of failure C_F. These costs are assumed to be independent of the time when they occur, i.e. no capitalization of costs is taken into account for convenience of presentation.

Under such conditions the expected total cost is given as

$$(10) \qquad C(t_I,t_s)=P_f'(t_I)C_F + [1-P_f'(t_I)]\cdot [C_I+\{P_R(t_I)\}\cdot\{C_R+P_f'''(t_s-t_I)C_F\}+ \{1-P_R(t_I)\}\cdot\{P_f''(t_s-t_I)C_F\}]$$

Because $P_f'(t_I)$ is increasing with t_I as well as $P_R(t_I)$, and $P_f''(t_s-t_I)$ as well as $P_f'''(t_s-t_I)$ are decreasing with t_I there must be a time t_I^* at which the total cost has a local minimum. Thus, an optimal value for the time to inspection can be determined.

The structural element is inspected at time t_I^* and actual observations become available on which a decision about repair is taken according to a prespecified rule. After repair at time t_I^* the structural element is treated as new. In case of no repair the crack state observed during inspection (with or without measurement error) is taken as the initial crack state for the analysis of the interval to a second inspection. Proceeding in this manner establishes an adaptive scheme for planning of times to inspection, which takes account of all available information.

The failure probability $P_f''(t_s-t_I)$ can be calculated by modifying $P_f'(t_s)$ under the condition that the structural element has survived up to time t_I

$$(11) \qquad P_f''(t_s-t_I) = P_f'(t_s) - P_f'(t_I)$$

This probability function of the time t_I to the first repair is not updated, because the crack length at time t_I is yet unknown. If, however, inspection is performed at time t_I a new failure probability function $P_f'_I(t-t_I)$ for time $t \geq t_I$ has to be calculated. Let $E(t_I)$ be the inspection event at time t_I, then the probability of failure at time of the first inspection and before any further inspection is calculated as

$$(12) \quad P_f'_I(t-t_I)=P[F(t-t_I)\,|\,E(t_I)]=\frac{P[F(t-t_I)\cap E(t_I)]}{P[E(t_I)]}$$

Nominator and denominator of the right hand side of this equation can be separately evaluated using modern reliability analysis software.

According to [Madsen 1982] it is useful to distinguish between two types of crack observations. Let $\hat{a}_i$ be an observed crack length and $A(t\,|\,r,q)$ the crack length at time t according to the calculation model (5). (Compare (6)ff for the meaning of "$|r,q$".) The observation events then are formulated as equality and inequality constraints

$$(13a) \qquad E_a(t_I) = \{(A(t_I\,|\,r,q)-\hat{a}_i = \epsilon_i)\}$$

where ϵ_i is the measurement error in the observation, or

$$(13b) \qquad E_{na}(t_I) = \{(A(t_I\,|\,r,q)-\hat{a}_b \leq \epsilon_b)\}$$

if no crack is observed. Asymptotic first and second order results for the evaluation of of probabilities of events with equality constraints were outlined the first time in [Madsen 1985] and studied more intensively in [Schall et al. 1988]. In (13a) $\hat{a}_b$ is the lowest detectable crack size and ϵ_b is the corresponding measurement error depending on the particular inspection method as before.

Whether a structural element is repaired after inspection or not depends on the result of the actual observation. The necessary decision rule can, in principle, be based on the potential gain in residual reliability. A simpler and probably more practical criterion directly uses the measured crack length. An example of such a crack length based decision rule D is given as

$$(14) \qquad D(t_I)=\begin{cases}\text{repair} & \text{if } A(t_I\,|\,r,q) \geq a_{c,rep}\\ \text{no repair} & \text{if } A(t_I\,|\,r,q) < a_{c,rep}\end{cases}$$

The critical repair crack length $a_{c,rep}$ needs to be predefined. Since the event of repair depends on the decision rule, the probability of repair is

$$(15) \qquad P_R(t_I) = P[R(t_I)]$$

where

$$(16) \qquad R(t_I) = \{a_{c,rep}-A(t_I\,|\,r,q) \leq 0\}$$

To further clarify the effect of repair (or replacement) the simplest model is to assume independence of the properties between the previous and the repaired condition of the structural element. The probability of failure after repair is then

$$P_f'''(t_s - t_1) = P_f'(t_s - t_1) \qquad (17)$$

Note that other repair strategies are possible and can also be incorporated.

EXAMPLE

Part of the outlined concept of adaptive planning of inspection and repair is now illustrated. The time interval to the first inspection is determined for a modern 3rd generation container ship which was analyzed in useful details in [Schall et al. 1991]. Some data of the vessel are collected in Table 1.

Table 1: Main data of Container Ship

L/m	B/m	H/m	D/m	Δ/to	V/m/s (av.)
157	23	11.8	8.9	20514	7.0

Stochastic Models of the Environment

The spectral moments m_i as used in (5) and (7) depend on stochastic variables H_v, T_v, ϑ. Suitable stochastic models for H_v and T_v can be inferred from long term seaway statistics that are available in form of scatter diagrams representing relative frequencies $p(H_v,T_v)$ of observed values of H_v and T_v. Based on published data [GLOBAL WAVE STATISTICS 1985], averaged over the different observation areas, e.g. 15/16/24/25 to include the main EAST–WEST shipping routes of the North Atlantic, the distributions of H_v and T_v were estimated in [Schall et al. 1991]

$$F(H_v,T_v) = F(T_v|H_v) \cdot F(H_v) \qquad (18)$$

with

$$F(T_v|H_v) = 1 - \exp\left[-\left[\frac{T_v - \kappa}{u(H_v)}\right]^{k(H_v)}\right] \qquad (19)$$

according to [Houmb & Overvik 1976], where

$$u(H_v) = a \exp[b\, H_v] \qquad (20)$$

$$k(H_v) = c \exp[d\, H_v] \qquad (21)$$

For simplicity, the parameter κ was taken independent of H_v. For the North Atlantic shipping route a maximum likelihood estimation determined e.g. $a = 5.7$, $b = 0.05$, $c = 4.6$, $d = 0.02$, and $\kappa = 2$, and a least square fit of the statistical data gave

$$F(H_v) = 1 - \exp\left[-\left[\frac{H_v}{2.71}\right]^{1.54}\right] \qquad (22)$$

Similar formulae were obtained for the Pacific shipping route.

The stochastic model of the angle ϑ between main direction of the seaway and the orientation of the structure was obtained in [Schall et al. 1991] as

$$F(\vartheta) = \frac{1}{2\pi}\vartheta \quad \text{for} \quad 0 \le \vartheta < 2\pi \qquad (23)$$

A deterministic value used for the still–water stress μ_S is given in Table 2.

Stochastic Models for Material Parameters

Material parameters should be modeled as random variables due to significant scatter of test data. Parameters such as fracture toughness K_{IC}, or yield strength S_y, and ultimate strength S_u, are modeled by a Weibull or log–normal distribution, respectively, see Table 2.

The initial crack length A_0 is mainly related to the specific material, method of fabrication process (cold worked, surface processes), type of connection (welded or bolted), and form of the structural element under consideration. In practice the choice of the distribution of the initial crack length raises one of the most difficult questions as it involves also techniques of quality control, inspection, and the definition of the termination of the crack initiation time. In Table 2 a reasonable choice for our purpose is presented.

The parameters of the crack growth law (4), i.e. the exponent M and the factor C, should also be modeled stochastically. A large number of laboratory and field data is available. M and C are necessarily highly and negatively correlated. For physical reasons, the exponent M appears to be nearly a constant with its value around $M = m = 4$. C is modeled by a lognormal distribution, see Table 2

Table 2: Stochastic Model and Parameter Assumptions

Parameter	Stoc. Model	Location Parameter	Dispersion Parameter
A_0	Rayl.	2 mm	–
M=m	–	4	–
C	Logn.	$5 \cdot 10^{-17}$	20%
K_{IC}	Weib.	100 $MNm^{3/2}$	15%
Y(A)	–	1.12	
S_y	Logn.	400 MPa	7%
S_u	Logn.	550 MPa	7%
H_v	Weib.	comp. (22)	comp. (22)
$T_v \vert H_v$	Weib.	comp. (19)	comp. (19)
ϑ	Unif.	0 – 2π	–
μ_S	Unif.	80 – 100 MPa	–

Spectral Moments of Stresses

Stresses at highly loaded locations (hot spots) of the vessel's structure are calculated by the finite element method. About 7000 nodal points and about 18000 elements were introduced to describe all strength relevant structural elements of the ship. Plain stress elements (about 14000) and truss elements (about 4000) were used. A few beam elements were needed to represent e.g. ship board cranes and the rudder. Six support elements were needed for free support, suppressing free body motions but not inducing any stresses in the ship.

The finite element code used is a special version of SAP IV [Bathe et al. 1974] as developed by Germanischer Lloyd. Figure 2 gives an impression of the FE model of the container ship.

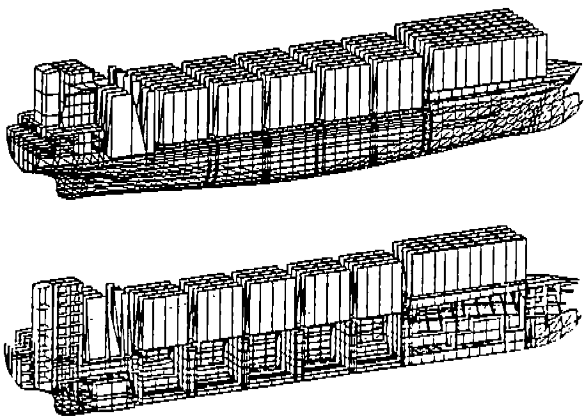

Figure 2: FE Model of a 3rd Generation Containership

The global stress analysis is too clumsy for accurately computing the stress concentrations and a detailed local FE model was developed to represent the neighborhood of the hot spot. Details of the FE–modeling of the hatch corner etc. are given in [Schall et al. 1991].

Local stresses could then be calculated for 40 different elementary wave periods $T=2\pi/\omega$ ranging from 0.75 s to 40.0 s with unit wave amplitude yielding the transfer functions of stresses $Y_{SZ}(\omega)$. With the transfer functions the stress spectra

(24) $$S_{SS}(H_v,T_v,\omega)=Y_{SZ}^2(\omega)\cdot S_{ZZ}(H_v,T_v,\omega)$$

could be established using parameters H_v and T_v to define S_{ZZ} as the Pierson–Moskwitz standard spectrum. The related three spectral moments m_0, m_2 and m_4 of S_{SS} as required in (5) and (7) were defined as

(25) $$m_i = {}_0\int^{\infty}\omega^i\cdot S_{SS}(\omega)d\omega$$

<u>Fatigue Failure Probability</u>

Evaluation of (9) was made possible by definition of response surfaces as quasi–analytical representations of the spectral moments m_i as functions of H_v, T_v and ϑ. The details are explained in [Schall et al. 1991].

Two service routes for the containership were considered, which differ significantly with respect to their wave climate. In Table 3 the fatigue failure probability is given implicitly as the so–called equivalent reliability index

(26) $$\beta_E(t_l) = -\Phi^{-1}(P_f'(t_l))$$

with Φ the standard normal distribution function. The higher β_E–values represent the lower P_f'–values. Table 3 indicates that there is a significant difference between the fatigue failure probability on different shipping routes. Thus, inclusion of statistical information on the wave climate as experienced by the ship on its actual route is one major concern of our further studies on adaptive inspection planning, which cannot, however, exemplified here.

Table 3: Location and Dispersion Parameter of the Significant Wave Height H_v, and Equivalent Reliability Index β_E for Different Sea Routes and Service Times

service route	service time	days at sea	Loc. par. H_v/m	Dsp. par. H_v/m	β_E
North Atlantik	4y	800	2.71	1.54	6.02
	8y	1600	2.71	1.54	5.97
	12y	2400	2.71	1.54	5.79
	15y	3000	2.71	1.54	3.89
Pacific	4y	800	2.12	2.01	7.62
	8y	1600	2.12	2.01	7.60
	12y	2400	2.12	2.01	7.47
	15y	3000	2.12	2.01	6.30

<u>Planning of Inspection and Repair</u>

In Figure 3 and 4 the calculated probability functions are given are given for the North Atlantic and Pacific shipping routes, respectively. Abbreviations are chosen as in Figure 1

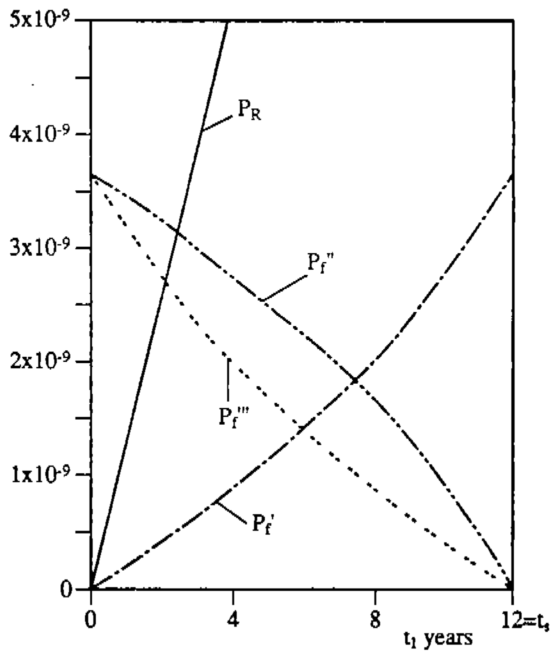

Figure 3: Probability Functions for the North Atlantic Shipping Route

$P_f'(t)$ is the failure probability during [0,t] of the structural element without inspection. $P_f'(t)$ and $P_R(t)$

are continuously growing with t while $P_f''(t)$ and $P_f'''(t)$ tend to zero for $t = t_s = 12$ years. Any other service time t_s would be a valid assumption, e.g. $t_s = 15$ years. Since greater service times had no significant effect on the optimal time to the first inspection t_1 but the appropriate graphical presentation of results became difficult, we used $t_s = 12$ years in the example.

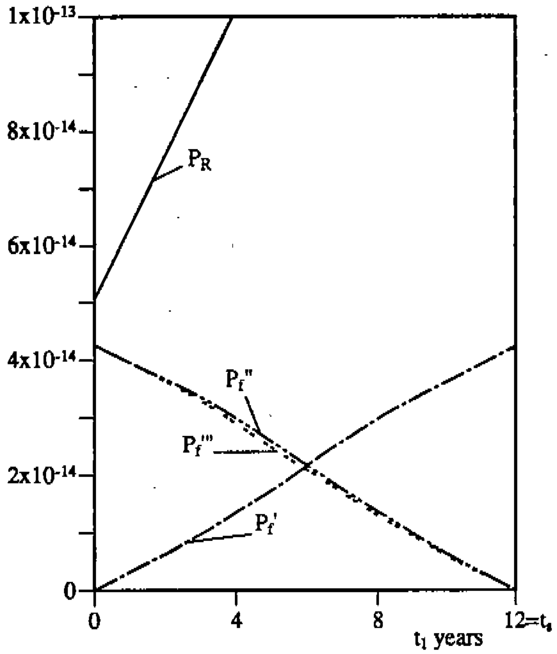

Figure 4: Probability Functions for the Pacific Shipping Route

Repair is performed if the crack size measured at time of inspection exceeds $a_{c,rep} = 15$ mm. Figure 4 indicates that there is always a small probability of repair P_R (i.e. there is a probability of a too large crack length according to (15) and (16)) even for a new ship with time to inspection $t_1 = 0$.

To evaluate (10) the following cost assumptions were taken:

$$C_I = 0 \qquad C_R = 10^6 \qquad C_F = 10^8$$

The inspection cost C_I is constant with time and comparatively low. C_I was set to zero in order to make possible a clear graphical presentation of probable cost functions in Figure 5 and 6, where the following abbreviations are used:

$$C' = P_f' \cdot C_F$$

(probable cost of failure before inspection)

$$C'' = [1-P_f'(t_1)] \cdot [\{1-P_R(t_1)\} \cdot \{P_f''(t_s-t_1)C_F\}]$$

(probable cost of no failure before inspection, but no repair after inspection and failure)

$$C''' = [1-P_f'(t_1)] \cdot [\{P_R(t_1)\} \cdot \{C_R+P_f'''(t_s-t_1)C_F\}]$$

(probable cost of no failure before inspection, but repair after inspection and failure)

$$C = C' + C'' + C''' + [1-P_f(t_1)] \cdot C_I$$

(expected total cost, i.e. sum of all probable costs, including probable inspection cost at t_1)

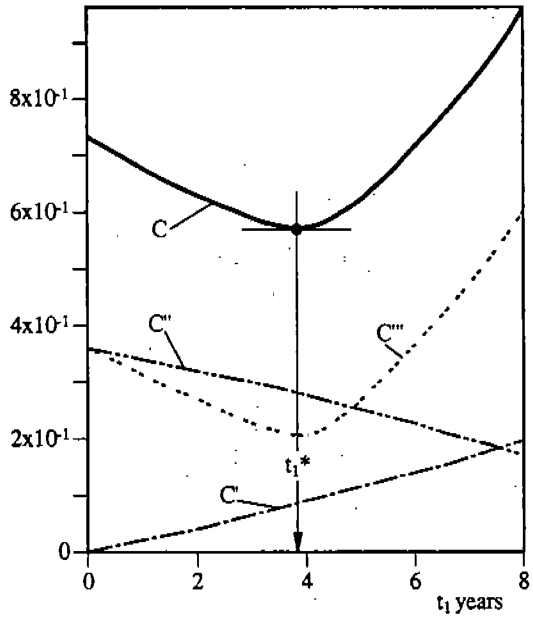

Figure 5: Probable Cost Functions for the North Atlantic Shipping Route

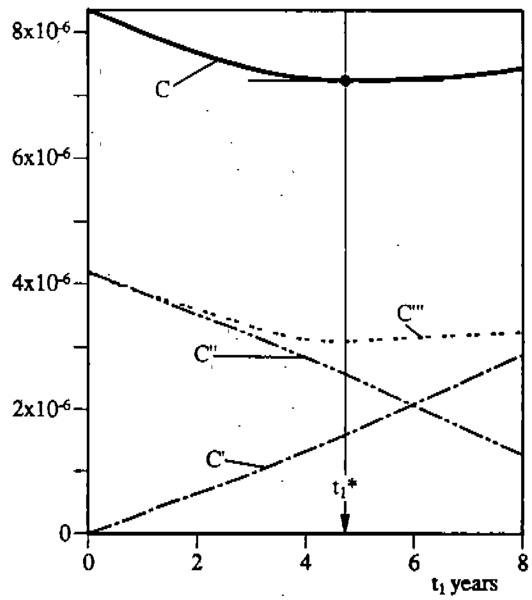

Figure 6: Probable Cost Functions for the Pacific Shipping Route

The expected total cost C increases with $t_1 > t_1^*$, which must be attributed to the strongly increasing probable cost functions $C'(t_1)$ and $C'''(t_1)$. Increasing the probable repair cost moves the minimum to the left, which strongly contributes to the result that the optimal inspection time for the North Atlantic Shipping route is shorter than for the Pacific. Increasing the repair threshold moves the minimum to the right because large cracks can only be observed at comparatively late times. Decreasing the inspection and repair cost also moves the minimum to the right.

According to Figures 5 and 6 first inspections would be optimal at $t_1^* = 3.8$ or 4.7 years, respectively. This result, if generalized, leads to the conclusion that first optimal inspection of the hatch corner could be delayed by nearly one year for continuous operation in the Pacific compared to continuous operation in the North Atlantic.

After the first inspection, various actions should be based on the decision concept that we recalled from [Fujita et al. 1989] in the previous chapter. These details were extensively discussed in [Fujita et al. 1989] and will not be repeated here.

CONCLUSION

A concept of adaptive inspection and repair planning based on a minimal cost principle has been presented and partially illustrated with the example of possible fatigue failure of a hatch corner of a modern container ship. The concept aims at rational decisions on inspection intervals and on the amount of inspections and repairs during a ship's service time. At present, further development is necessary and under way. Therefore, we do not suggest immediate application but we think that such rationalization of inspection strategies with application of modern methods of reliability analysis will be adopted to effectively control practical ship operations.

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Walter MacLean

May I offer the suggestion that you have just made the case for monitoring systems and by monitoring the environment as well as the response in key areas you can develop a rational approach for inspection. Would this be a safe projection?

C. Östergaard

Yes, I think so. It's one of the aims that we have in mind.