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The effect of corrosion on the structural reliability of steel offshore structures

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Abstract

This paper considers essential theoretical concepts and data requirements for engineering structural reliability assessment suitable for the estimation of the safety and reliability of corroding ships, offshore structures and pipelines. Such infrastructure operates in a harsh environment. Allowance must be made for structural deterioration since protective measures such as paint coatings, galvanizing or cathodic protection may be ineffective. Reliability analysis requires accurate engineering models for the description and prediction of material corrosion loss and for the maximum depth of pitting. New probability-based models for both these forms of corrosion have been proposed recently and calibrated against a wide range of data. The effects of water velocity and of water pollution are reviewed and compared with recently reported field data for a corrosion at an offshore oil platform. The data interpreted according to the model show good correlation when allowance is made for the season of first immersion and the adverse effects of seawater velocity and of water pollution. An example is given to illustrate the application of reliability analysis to a pipeline subject to pitting corrosion. An important outcome is that good quality estimation of the longer-term probability of loss of structural integrity requires good modelling of the longer-term corrosion behaviour. This is usually associated with anaerobic corrosion. As a result, it cannot be extrapolated from data for short-term corrosion as this is associated with aerobic corrosion conditions.

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1. Introduction

For offshore and land-based structural systems (structures, pipelines) and for shipping the deterioration of structural strength and structural integrity is a major factor in assets management. Modern approaches to asset management increasingly desire quantitative assessments. These include estimates of the probability of safety within a given remaining life or estimation of remaining life for a given level of performance. These matters are influenced by the loss of section thickness for structural elements and by the potential for loss of integrity through pitting corrosion. It is particularly critical where protective measures, such as paint coatings, galvanizing or cathodic protection, are or may become ineffective. The quantitative approach commonly employed for assessing the remaining safe life of infrastructure is modern structural reliability theory, founded on accepted mathematical concepts [1–3].

For probabilistic methods to be applied to their best advantage requires good quality stochastic models for the loadings (natural loads such as wind, wave, earthquake and man-made loads) applied to a structure. There is wealth of literature in this area. Also, probabilistic models are required for modelling the strength of materials and of dimensions. Here, too, there is much information available in the literature, including for time-dependent effects such as fatigue [3].

The most serious current deficiency is the availability of sufficiently accurate probabilistic models for corrosion, both for loss of material and for pitting depth as functions of time. The information available to structural engineers about corrosion tends to be anecdotal, not well organized and of limited use even for simple applications [4]. While there is a long and very considerable history of corrosion research, much of this is focussed on elucidating fundamental scientific principles, often using laboratory observations. Applied corrosion research is largely empirical, attempting to develop correlations between field observations of corrosion and environmental effects, most notably for atmospheric corrosion [5,6]. These efforts have not yet yielded models that are capable of predicting corrosion losses or pit depths independent of local calibration. Nor are they necessarily very accurate. For reasonably accurate quantitative engineering assessment purposes, better, predictive, models are highly desirable. These models must account for the inherent uncertainties associated with corrosion processes. They must also be able to deal with uncertainties in environmental conditions.

In practice, predictive methods may be supplemented by observations from indirect methods, such as corrosion potentials and impedance measurements. Usually these are difficult to apply under field conditions and require access to the region of interest. In addition, there is the problem of extrapolating from highly localized data points to larger structural elements (such as ships and offshore structures). Nevertheless, any information they supply can be used with 'Bayesian up-dating' to supplement a reliability estimate [3].

The present paper outlines basic concepts and approaches in time-variant structural reliability analysis and its application in engineering risk assessment and in asset management. Attention is then turned to a description of recently developed probabilistic, phenomenological models for general corrosion loss and for pitting corrosion. The influences of water velocity and of water pollution as might be associated with offshore oil platforms are described. To illustrate the concepts involved a simple example application for pitting corrosion of a pipeline is given, showing the trend increase in failure probability with exposure time.

2. Structural reliability estimation [1–3]

There are two key groups of input elements for structural reliability estimation—the loads acting on the system and the capacity (resistance) of the system to support them. For structures at sea, for example, the loads principally will be forces generated due to wave action and ship motion. Such forces will be generated globally, such as those leading to hull girder bending, and locally, such as those at side and bottom panels, at hatches, etc. The forces will be a function of the wave height and hence sea-state, and can arise also from wind conditions. There may be effects also from differential temperatures and from cargo load conditions. Each applied load can be represented as a stochastic process $Q(t)$ having a probability density function $f_Q(q(t))$. Here upper-case letters represent random variables and lower-case deterministic (non-random) variables. Also t is time. A typical realization of a load process is shown in Fig. 1. In general there will be several possible loading systems acting on a structure. They may be represented by the stochastic load vector $\mathbf{Q}(t)$.

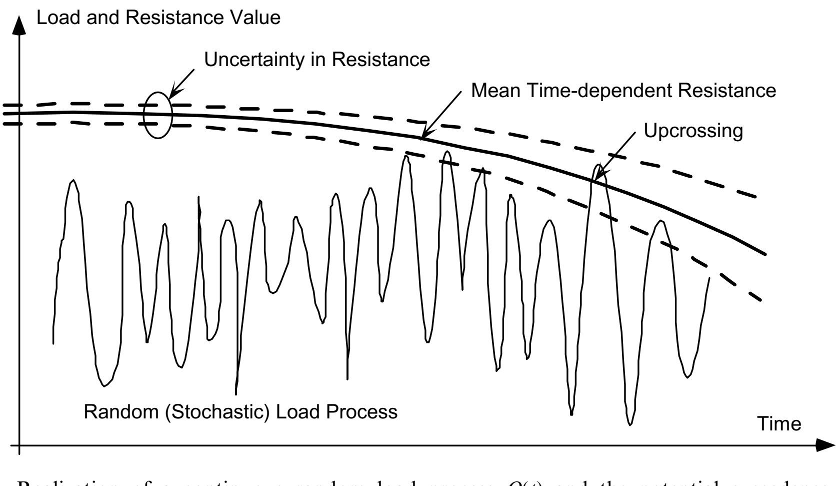

Fig. 1. Realization of a continuous random load process $Q(t)$ and the potential exceedence of the deteriorating structural resistance $R(t)$.

The other key group of elements in a reliability analysis are the variables describing the resistance of the structure to applied loading. Typically these are random variables and some may change (deteriorate) with time. Thus, the strength may be denoted $R(t)$ indicating its time dependence. Its probabilistic nature (uncertainty) is represented by the probability density function $f_R(r(t))$. Again, in realistic structures there will be many resistance variables. They may be represented by the vector of random strengths $\mathbf{R}(t)$. Fig. 1 shows the time-dependent deterioration of just one component of $\mathbf{R}(t)$.

The relationship between the loadings and the structural strength typically is given by one or more limit state functions written in general format as $G[\mathbf{Q}(t), \mathbf{R}(t)] = 0$. There also may be similar expressions for other design requirements such as deflection limits. For the simplest case with one loading process and one deteriorating resistance, the limit state function becomes $G[Q(t) - R(t)] = 0$ with $G < 0$ denoting violation of the limit state or 'failure'. This is seen in Fig. 1 at $t = t_1$ when $Q(t_1) > R(t_1)$. Note that in general there will be uncertainty both about the precise strength and the rate of strength loss with time, as shown schematically in Fig. 1.

Following directly from these concepts, the probability that a structure will fail in a given time period $[0, t_L]$ can be stated as the probability that the structure will fail when it is first loaded, denoted $p_f(0, t_L)$, plus the probability that it will fail subsequently, given that it has not failed earlier. This can be expressed as:

$$p_f(t) \approx p_f(0, t_L) + [1 - p_f(0, t_L)] \cdot [1 - e^{-\nu t}] \tag{1}$$

where ν is the so-called 'outcrossing rate'. It is the average rate at which the load vector $\mathbf{Q}(t)$ would be expected to leave the safe region defined by the limit state functions for given realization of $\mathbf{R}(t) = \mathbf{r}(t)$. See Fig. 2. To account for the variability of $\mathbf{r}(t)$ integration over all $\mathbf{r}(t)$ is the next required step. Evidently, this is a major exercise for $\mathbf{r}(t)$ of large dimensions.

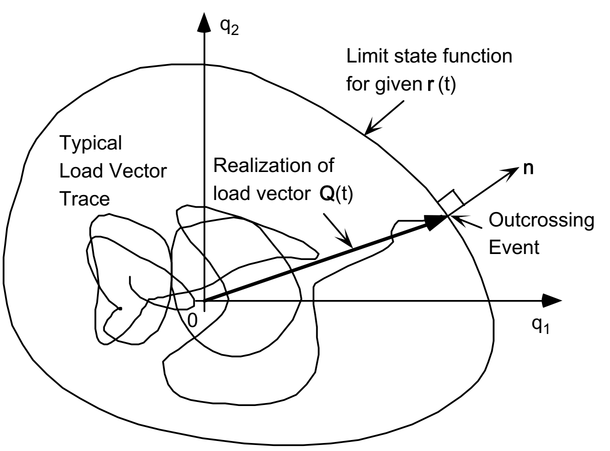

Fig. 2. Typical outcrossing event in structural resistance space $\mathbf{r}(t)$.

Expression (1) is shown as approximate because the second square bracketed [] term as written is based on the assumption that failure events are 'rare' and that such events therefore can be represented by a Poisson distribution (as shown in the last term). (More general forms of (1) are available for higher probabilities but at considerably greater complexity.)

The probability of failure can be estimated from (1) provided the outcrossing rate can be estimated. The most common approach is to assume that each random load process $Q(t)$ continues indefinitely and has a 'stationary' statistical nature (e.g. in the simplest case, the means and variance do not change with time). When there are several such loads, the rate at which they would be expected to 'cross-out' of the safe domain can be estimated from:

$$v = \int_{\text{safe domain}} E(\dot{X}_{\mathrm{n}}|\mathbf{X} = \mathbf{x})^{+} f_{\mathbf{X}}(\mathbf{x})\,\mathrm{d}\mathbf{x} \tag{2}$$

where $\mathbf{X} = \mathbf{X}(t)$ is a vector process (usually but not always restricted to $\mathbf{Q}(t)$), $\dot{X}$ is the time derivative of $\mathbf{X}(t)$ and $()^{+}$ denotes the positive component only. The term $E(\dot{X}_{\mathrm{n}}|\mathbf{X} = \mathbf{x}) = \dot{x}_{\mathrm{n}} = \mathbf{n}(t) \cdot \dot{\mathbf{x}}(t) > 0$ represents the outward normal component of the vector process at the domain boundary (Fig. 2). This result can be extended readily to allow for gradual deterioration of structural strength with time. In this case both v and $p_{\mathrm{f}}(0, t_{\mathrm{L}})$ become time dependent. As the structural resistance decreases the Poisson approximation used for (1) becomes gradually less applicable and for accurate estimates more complex techniques must be used [3]. It follows that accurate representation of deterioration is important.

Since the above formulation is not always easy to apply, in practice simplified and asymptotic methods are often substituted. The most obvious is to direct application of a Monte Carlo technique to the above formulations analysis but this is extremely computer-time demanding. A more common alternative is to make the problem 'time-invariant'. To do this each load process is replaced by a random variable chosen to be equivalent to the uncertainty associated with the largest load value in a given time horizon (say the design life). This allows each load to be represented by an extreme value distribution. Since combinations of loads acting on a structure could not usually all be at their extreme value at the same time, special allowance has to be made for load combinations. A practical approximation is 'Turkstra's' rule [3]. Where there is only one load applied to the structure load combination rules are not necessary. For simplicity this will be the case assumed in the example to be discussed later.

Once the problem is reformulated as a time-invariant case, simpler structural reliability techniques can be applied. These are described next. How these can deal with structural deterioration will then be described briefly and through an example at the end of this paper.

Time-invariant methods estimate a probability of failure p_{T} at any time t from [3]:

$$p_{\mathrm{T}} = \int_{G(\mathbf{x})<0} f_{\mathbf{X}}(\mathbf{x})\,\mathrm{d}\mathbf{x} \tag{3}$$

where $f_{\mathbf{X}}(\mathbf{x})$ is the joint probability density function for the vector of random variables $\mathbf{X}$ (which now incorporates $\mathbf{Q}(t)$ as random variables and $\mathbf{R}(t)$ at a given time t). Here the failure region is described by $G(\mathbf{x}) < 0$ with the function $G(\mathbf{x}) = 0$ being the limit state function, as before.

The multiple integral (3) may be solved approximately using numerical integration. There is a wealth of literature dealing with the numerical solution of multiple integrals—ordinary numerical methods become excessively demanding for problems with $\mathbf{X}$ of large dimensions and for these Monte Carlo tends to be better suited. This is also a numerical approximation. It has very considerable power to deal with any probability distributions for the $\mathbf{X}$ and with complex (and multiple) limit state functions [3].

An alternative to numerical approximation is the use of asymptotic methods, most well-known being the FOSM/FORM time-invariant techniques [3]. Here FO denotes 'First Order' implying that the limit state function is approximated by a linear function. 'SM' denotes 'Second Moment' or representation of all random variables by their means and standard deviations (second moment) only. For some simple problems this may be appropriate but a refinement is to approximate the actual probability functions in $\mathbf{X}$ in the 'tails' of interest by a Normal (Gaussian) distribution and to use an iterative process. This has been termed the First Order Reliability Method (FORM). Details of the calculation procedure are available in standard texts and in software packages [3] and need not be described here.

A time-invariant analysis carried at time $t = t_i$ assumed independence of the analysis at any other time $t = t_j$ ($j \neq i$). This is not strictly correct, since the resistances $\mathbf{R}(t)$ at different points in time are not independent. In fact they are essentially the same (random quantity), except for deterioration. The error in this approximation (the so-called 'ensemble average' approximation) has only recently been studied systematically [7]. Nevertheless, it is a convenient approximation and one that will be used in the example below.

A number of authors have introduced corrosion into structural reliability analysis through using either a simplification of the time-variant approach or through the simplified ensemble average approach. In most cases further simplifications were introduced, rendering the estimated failure probabilities 'nominal'. Applications have been given for ships and offshore structures [8–14]. Because of the complexity of the problems, all used simplified reliability analysis procedures. They also used empirical information on corrosion rates.

In all cases the corrosion models were based on data pooled from a variety of sources without regard to differences in operating conditions, environments and the ages of the ships involved [15–17]. As a result, the corrosion data showed a high degree of scatter and the models constructed on them show a high degree of uncertainty, with coefficients of variation in the range 0.5–2.0 [18]. Unfortunately, corrosion models based on pooled data have no validity for predicting the likely future condition for a particular ship or structure. This has lead to the development of better, ship specific models [19].

3. Effect of corrosion on structures

For structural systems such as ships, offshore platforms, pipelines and pressure vessels there usually are two critical design criteria—strength capacity and integrity. The first is essentially a function of the amount of material loss due to surface or general corrosion (although it may be affected by localized corrosion). The second is essentially localized and in particular due to pitting corrosion. Each of these will now be described from a structural capacity perspective.

Typically structural capacity depends primarily on the cross-sectional dimensions of a structural member. For example, for an axial member of cross-sectional area A under axial stress σ and surrounded by seawater the capacity is given by (Fig. 3a)

$$R(t) = \sigma[A - P \cdot c(t)] \tag{4}$$

where $c(t)$ is the corrosion loss as a function of time and P is the perimeter area exposed to seawater. For plates in bending, with corrosion possible on each side of the plate, the bending resistance becomes

$$R(t) = k \cdot \sigma_b \cdot [d(t)]^2 = k \cdot \sigma_b \cdot [d_0 - 2 \cdot c(t)]^2 \tag{5}$$

where σ_b is the maximum (extreme fibre) stress imposed by bending action, $d(t)$ is the remaining thickness and d_0 is the initial thickness of the plate and $k = 0.25$ for elastic–plastic (i.e. ductile) material response and 0.167 for elastic (i.e. brittle) response (Fig. 3b).

For strength considerations in ductile (plastic) structures it is sufficient to obtain $c(t)$ from weight-loss measurements or estimates of the corrosion depth, averaged over a local surface area. Where local stress intensity may cause local rupture, as in brittle materials, the local maximum for $c(t)$ at the point of maximum local stress is required.

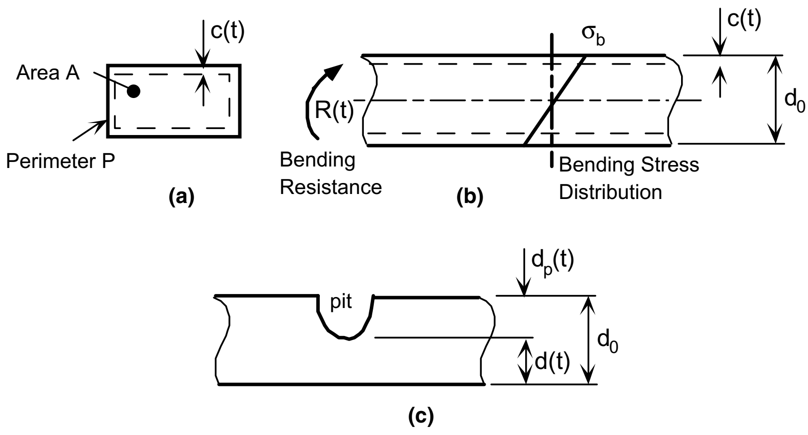

Fig. 3. (a) Cross-section of a bar under axial stress showing corrosion loss, (b) cross-section of a plate under bending stress showing effect of corrosion and (c) cross-section of a plate subject to pitting from one side.

For pitting the remaining thickness $d(t)$ of a plate of initial thickness d_0 is given by (Fig. 3c)

$$d(t) = d_0 - d_p(t) \tag{6}$$

where $d_p(t)$ is the maximum pit depth at time t. The maximum probable pit depth is required since the first (deepest) pit will cause first perforation. This occurs when $d(t) \to 0$.

In practice the dimensions such as d_0 will vary somewhat from location to location over the surface of a steel plate or member, and in addition the actual dimension may differ appreciably from the nominal dimensions. This gives rise to variability and to bias respectively for the dimensions. A considerable amount of data for this is available [3]. The locally relevant applied stress (σ or σ_b) depends directly on the loadings applied to the structure and hence also have random properties (expressed either as random variables or random processes).

That there is considerable variability in corrosion losses such as $c(t)$ and $d_p(t)$ is evident from the wide disparity in average corrosion rates ($c(t)/t$ and $d_p(t)/t$) quoted in the literature [4,20,21]. It has been suggested previously that a more appropriate approach is to consider these parameters as random variables with properties that change with time [22]. Thus it is appropriate to write

$$c(t, \mathbf{E}) = b(t, \mathbf{E}) \cdot fn(t, \mathbf{E}) + \varepsilon(t, \mathbf{E}) \tag{7}$$

where $c(t, \mathbf{E})$ is the corrosion loss of material, $fn(t, \mathbf{E})$ is a mean valued function, $b(t, \mathbf{E})$ is a bias function, $\varepsilon(t, \mathbf{E})$ is a zero mean error function (a probability 'bucket') and $\mathbf{E}$ is a vector of environmental (and material) parameters. A similar formulation can be applied for $d_p(t)$. The discussions in the next section focus primarily on $fn(t, \mathbf{E})$.

Most corrosion data available to engineers is quoted in terms of a corrosion rate [4,20] despite fundamental theoretical considerations that corrosion should decrease with time [20]. Early attempts to account for the known non-linearity have been reviewed [23] but only recently has an attempt been made to develop engineering models based on corrosion science principles [24–26]. This work has so far been confined to marine immersion corrosion of structural steel, both general corrosion and pitting. It is currently being extended to tidal and coastal atmospheric corrosion. An important component for these models is their consideration also of bacteriological influences [24]. The models are based on coupon data, with the reasonable assumption that this can be extrapolated to larger plate areas.

4. Model for general corrosion of structural steels

4.1. 'At-sea' model

For immersion corrosion of mild or low alloy structural steels, the model shown in Fig. 4 was proposed earlier [27]. Its parameters have been calibrated to a wide range of field data and estimates made of variability [24]. Although initially termed

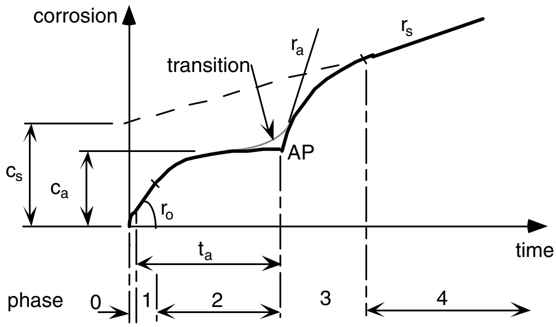

Fig. 4. Schematic representation of the changing phase in immersion corrosion loss as a function of exposure time [24].

Table 1
Phases and calibrated parameters for $fn(t, T)$ as a function of T for general corrosion model

Phase	Phase description and corrosion controlling mechanism	Governing parameters as function of T	Correlation coefficient
0	Short-term initial corrosion governed mainly by chemical kinetics	–	
1	Approximately linear function governed by oxygen diffusion from surrounding water without inhibition from corrosion products and marine growth	$r_0 = 0.076\exp(-0.054T)$	$R = 0.963$
2	Non-linear function governed by oxygen diffusion through corrosion product layer of increasing thickness	$t_a = 6.61\exp(-0.088T)$ $c_a = 0.32\exp(-0.038T)$	$R = 0.99$ $R = 0.944$
3	Anaerobic bacterial corrosion phase governed by transport and diffusion of energy sources stored in corrosion product to corroding interface	$r_a = 0.066\exp(0.061T)$	$R = 0.97$
4	Approximately linear long-term anaerobic bacterial corrosion phase	$c_s = 0.075 + 5678T^{-4}$ $r_s = 0.045\exp(0.017T)$	– $R = 0.71$

a ‘phenomenological’ model, implying that it was based only on observations of field data, increasingly it is being related to fundamental corrosion science concepts, with phases 1 and 2 having already been represented by mathematical models dealing with oxygen transport [26,28]. Models of phases 3 and 4 are currently being developed.

The phases shown in Fig. 4 are summarized in Table 1. Each represents a different mechanism that controls the (instantaneous) rate of corrosion. An important finding is that for nominally ‘at-sea’ conditions for the major seawater bodies, environmental variables constituting **E** in (7), such as salinity, marine growth, dissolved oxygen, wave action and pollution are either very tightly controlled (such as through buffering action), closely similar or absent. It has been argued earlier [24] that the main variable for 'at-sea' conditions is annual average seawater temperature T. Calibration to over 16 data sources for both low carbon steel and copper-bearing steels showed that the parameters in the model (Fig. 4) could be formulated as in Table 1. Data for the variability of corrosion loss as a function of time was obtained in specially conducted experiments and has been estimated for variability between coupons and between locations [25].

Evidently, this model differs very considerably from the bi-logarithmic model $c(t) = A \cdot t^B$ typically adopted to describe corrosion loss as a function of time of exposure (where A and B are constants obtained empirically) [5,6]. Such a relationship is based on relatively indiscriminate use of empirical observations. It is also based in part on early considerations of the oxidation of metals [23], assuming that oxidation continues indefinitely. However, careful assessment of the empirical data and the observations described by various investigators supports the existence of the anaerobic phases 3 and 4 [24].

4.2. Effect of water velocity

The effect of water velocity is of practical interest. The classical and widely accepted work in this area is by LaQue [30]. He reported that the corrosion rate of steel increased with increased velocity but at an exponentially decreasing rate, such that there was little addition effect for water velocities greater than about 6 m/s. These observations were based on tests of 36 days duration in a recirculating system. This renders their validity for longer durations under actual field conditions problematic. A recent study [29] has shown that the effect is somewhat more complicated and that it cannot be represented simply by a changing corrosion rate. Fig. 5 shows

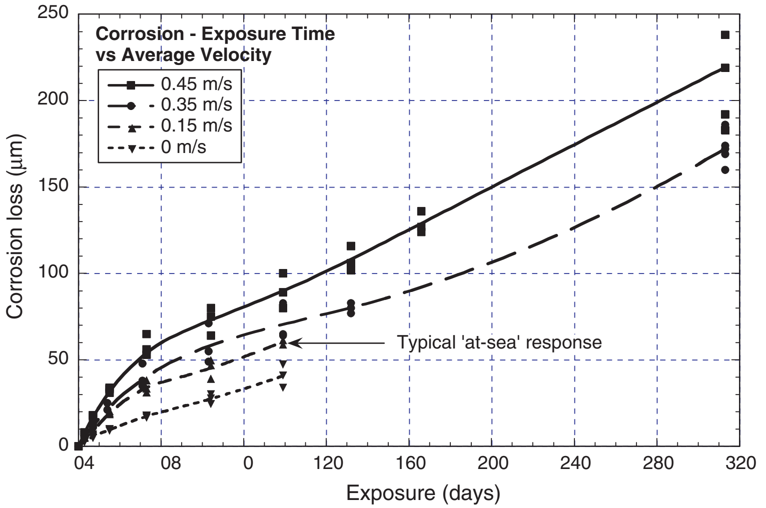

Fig. 5. Effect of water velocity on early corrosion loss.

typical results for seawater at 20 °C average temperature. It is seen that velocity has a profound effect during the early period of exposure. After this there is only a net effect resulting from the early period exposure with corrosion loss increased roughly by a constant amount at all later periods. In essence, as a result of velocity, the corrosion–time curve is translated 'upward' by a constant amount after the initial period of corrosion, with the amount of translation depending on the velocity.

As yet there are no substantially similar results for other water temperatures. However, it might be expected that due to the faster development of a protective corrosion product layer and more marine growth at higher water temperatures, the effect of velocity would be somewhat lower and vice versa for lower temperatures [31]. At this stage there are no estimates of the uncertainties involved in relationships such as shown in Fig. 5.

4.3. Effect of nutrient pollution

A preliminary indication of the effects of pollution can be obtained simply from considering the influence of the environment on the phases of the model shown in Fig. 4. Evidently, depressed dissolved oxygen (DO) levels will reduce the rate of the diffusion of oxygen in phases 1 and 2 [24,26]. However, DO is unlikely to have any effect on phases 3 and 4 as corrosion in these phases is controlled by anaerobic conditions.

A rather unexpected set of field observations led to the conclusion that nutrient pollution, such as resulting from agricultural run-off or sewage pollution in coastal regions and waste waters and oil pollution at offshore oil fields, can affect phases 3 and 4. Test results obtained at two closely similar sites on the eastern Australian seaboard are shown in Fig. 6. Evidently, the corrosion observed was similar in the early (aerobic) corrosion phases, but diverged considerably later. Water quality testing revealed that site B was situated in waters with high levels of agricultural pollutants (nutrients). An examination of apparently anomalously high corrosion observations for several other sites around the world supports the possibility of nutrient pollution having been a factor [32]. Although the effect seems to be clear qualitatively, at this stage the relationship(s) between nutrient levels and increased corrosion has not been quantified.

4.4. Example—immersion corrosion in Chengdao oil field

Li et al. [33] reported field data for general corrosion at the transport wharf for the Shengli oil field, located in the offshore oil exploitation area in the Chengdao Sea. The observations extended over a 3-year period and involved three types of steel (A3, 16Mn and 20#). Corrosion losses were reported as time-dependent rates [33]. Conversion to corrosion loss produces the data points shown in Fig. 7 for each of the three steels. As might be expected for corrosion controlled by oxygen diffusion [23,34], there is little difference between the corrosion loss reported for the three steels during the first 2 years since this period corresponds with phases 1 and 2. Only

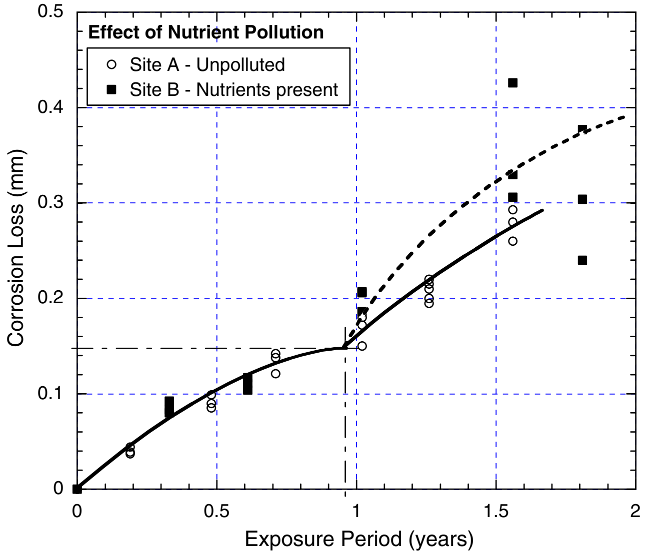

Fig. 6. Corrosion loss—period of exposure curves for two similar sites, with site B found to have a high nutrient level.

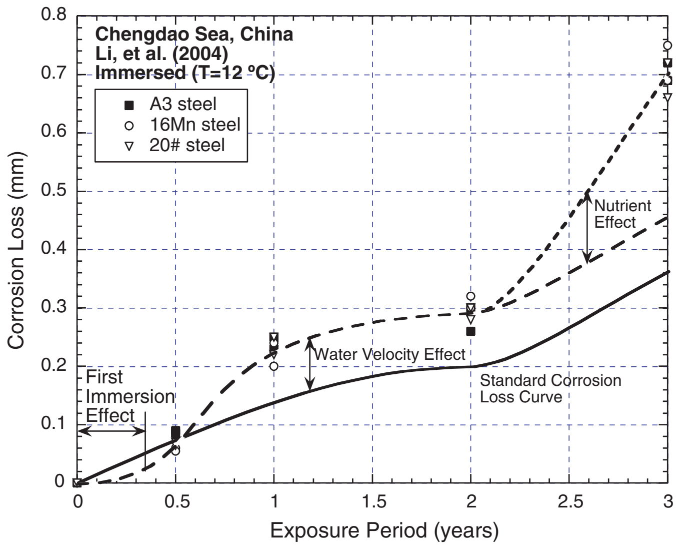

Fig. 7. Observed data for corrosion loss of three different steels, interpreted trend line and comparison to the standard corrosion loss curve for seawater estimated at 12 °C.

at year 3 is there much greater scatter in results. Moreover, it is evident that, overall, the points are consistent with the trend predicted by the model of Fig. 4.

The authors did not report the average seawater temperature. From world-wide sea surface temperature maps it is estimated to have been about 12 °C. This allows the relationships in Table 1 to be used to estimate the expected 'standard corrosion loss curve' shown in Fig. 7. The time of first immersion was such that an initial (winter) period of low corrosion was likely [35]. In view of the observation [33] that 'the highest (water) velocity is up to 1.5 m/s the velocity effect is likely to be as shown. Unfortunately the information available is insufficient to relate this quantitatively to Fig. 5.

Fig. 7 also shows a higher than expected amount of corrosion after about 2 years exposure. In view of the location of the test site within an operating oil field, nutrient pollution is not unlikely, as suggested also by the trend shown in Fig. 7. As noted, present understanding of the relationship between increased corrosion and nutrient levels is insufficient to provide quantitative comparisons.

5. Model for pitting corrosion of structural steels

For ships, offshore structures and pipelines the integrity of the system against perforation is also an important design and maintenance issue. Following on from the model for general corrosion, a refined model (Fig. 8) has been proposed for pitting corrosion [36]. It is based in part on in situ test observations and in part on theoretical concepts. The effect of anaerobic bacterial activity is even more evident in plots for maximum pit depth as a function of time, obtained from in situ field tests [36,37]. The data base is less extensive than that for general corrosion but it was sufficient to calibrate the parameters of the model (Table 2). In this case there is an early period to allow for the laboratory observations [38,39] that pitting initiates almost immediately on first exposure. Pits are observed to interact with general corrosion and appear to be involved in the continually changing morphology of the corroded surface [40].

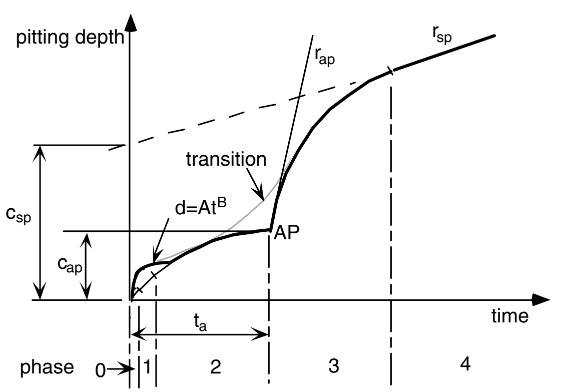

Fig. 8. Model for maximum pit depth as a function of exposure period, showing corrosion phases [36].

Table 2
Phases in pitting corrosion and calibrated functions for model parameters [36]

Phase	Phase description and corrosion controlling mechanism	Governing parameters as function of T
0	Initial pit growth	
1 and 2	Pit growth under overall aerobic conditions under rust cover	$t_a = 6.61\exp(-0.088T)$ $c_{ap} = 0.99\exp(-0.052T)$
3	Rapid pit growth under overall anaerobic conditions under rust cover	$r_{ap} = 0.596\exp(0.0526T)$
4	Steady-state pit growth under overall anaerobic conditions under rust cover	$c_{sp} = 0.641\exp(0.0613T)$ $r_{sp} = 0.353\exp(-0.0436T)$

Using the results from multiple coupon tests it is possible to estimate the variability in the depth of maximum pits as a function of time, comprehensively for 20 °C and approximately for other temperatures [37]. Fig. 9 shows an example.

It is evident from Fig. 8 that early pitting depth is of little interest for long-term maximum pit depths. As in Fig. 4 these are governed by anaerobic bacterial activity. Conventional considerations of oxygen transport and electrolyte characteristics are not involved [41]. This means there is no direct link between early extreme pitting behaviour and longer-term behaviour. As a result there is little merit or justification in applying the conventional pit growth models of the type $d(t) = A \cdot t^B - t_i$ (where A and B are constants obtained and t_i is the time before first pitting, all estimated empirically). This recognition has important practical implications, as will be seen in the example below.

It would be expected that as for general corrosion, pitting corrosion as a function of time will be influenced by water velocity, DO levels, nutrient load and season of first immersion. Unfortunately, it appears that for low alloy structural steels such

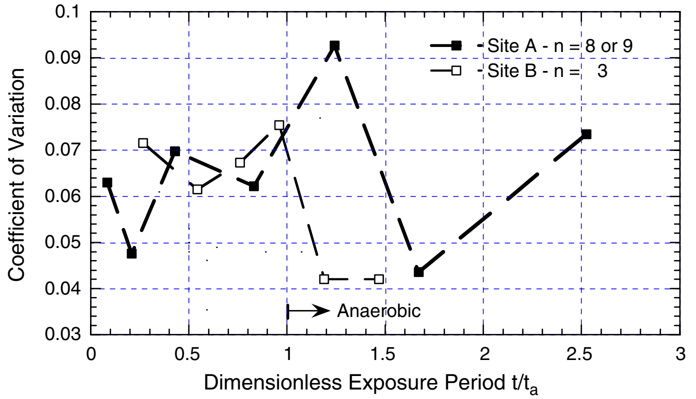

Fig. 9. Estimated standard deviation for pitting variability.

relationships (and in some cases the phenomena) have not yet been elucidated. Nevertheless, it is considered that these are likely to be important factors in practice.

6. Example of structural reliability estimation for pitting corrosion

Consider the case of a pipeline of wall thickness 8 mm exposed to external seawater at an average temperature of 10 °C. Suppose the protective coating has been rendered ineffective. Also suppose that the limit states for strength are not of interest at this stage. The question of interest is the probability of loss of integrity (i.e. loss of containment) for the pipeline as extreme pit depths increase with age. If d_0 is the pipeline wall thickness, containment is lost when $G = d_0 - d(t) = 0$ where $d(t)$ is the extreme pit depth.

The relationship for expected extreme pit depth $d(t)$ as a function of exposure time t was determined using the parameters in Table 2. To operationalize this, a function was fitted to each phase. This allowed the limit state function to be expressed as three component limit state functions as follows:

$$\text{Phases 1 and 2} \quad G_{1,2} = d_0 - a \cdot t^b = 0 \quad \text{for } 0 < t < t_a \tag{8a}$$

$$\text{Phase 3} \quad G_3 = d_0 - [c(t - t_a)^d + c_a] = 0 \quad \text{for } t_a < t < t_3 \tag{8b}$$

$$\text{Phase 4} \quad G_4 = d_0 - r_s \cdot t - c_s = 0 \quad \text{for } t > t_3 \tag{8c}$$

where (a, b, c, d, t_3) are constants obtained by fitting (8) to the pit depth curve. Table 3 lists the principal values obtained. Since the corrosion–time relationship is univalent, it is sufficient to write the limit state function as being simply $G = \max[G_{1,2}, G_3, G_4]$ without specific consideration of the time periods when each component limit state function is applicable. Table 3 also shows standard deviations estimated from the data [37] and Fig. 8. Moreover, the probability distributions considered most relevant for representing the variability of each of the variables is shown. For illustration, the variability of the original wall thickness was assumed to be Log–Normal with a standard deviation of 0.5 mm.

The structural reliability problem was analysed using time-invariant theory. The probability of failure was estimated as independent events, for different point in time, using a commercially available software package [42]. As noted above, it is not

Table 3
Statistical properties of parameters used in example

Parameter	Mean value	Standard deviation	Distribution
t_a (model parameter)	2.7 years	–	–
c_a (model parameter)	0.6 mm	–	–
a	0.4	0.1	Normal
b	0.7	0.175	Normal
c	0.81	0.1	Normal
d	0.59	0.07	Normal
c_s	1.0	0.1	Normal
r_s	0.25	0.05	Normal

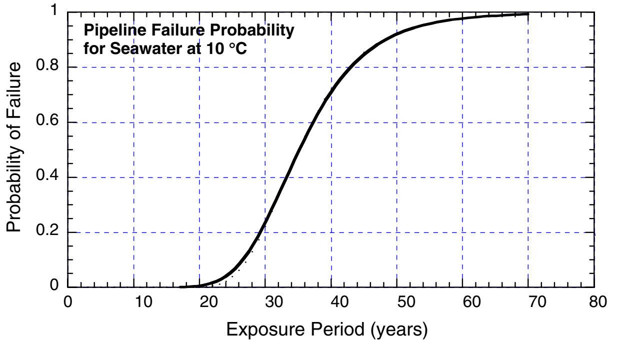

Fig. 10. Estimated probability of failure for pipeline example.

strictly correct to compare the probabilities with time, but any one probability estimate is a valid estimate for the probability of failure at that time point without the knowledge of (any) previous estimates. [To be precise, if the probability at some previous time(s) is known, the later probability estimate must be estimated conditional on the earlier one(s).]

The development with increased exposure time of the independent estimates of failure probability is shown in Fig. 10. Examination of the calculation procedure supported the expectation that phase 4 is responsible for the higher failure probabilities and that it becomes more prominent for longer exposure periods. Moreover, it follows easily that this is governed mainly by the parameter r_s. Phases 1 and 2 are much less important.

Fig. 10 shows that there is a considerable increase in the probability of failure after about 20 years exposure. For maintenance strategies this suggests that pipeline inspection should be increased at about this time. Such inspection may reveal whether the expected loss of wall thickness has actually occurred and this information may then be used in a Bayesian up-dating scheme to refine the estimates of failure probability for longer periods of use. Details of such procedures are not central to the discussion here and are available in the literature [3].

Phases 3 and 4, of which phase 4 is crucial to the probabilities in Fig. 10, are associated with anaerobic bacterial activity. As in the general corrosion loss model, this occurs after oxidation phases 0–2 and so is somewhat dependent on them but not related in terms of corrosion process. As a result, pit depth models of the type $A \cdot t^B$ and commonly calibrated by fitting to short-term observations, are of doubtful validity for estimating long term pitting behaviour.

7. Discussion

The above example illustrates why good quality models for corrosion behaviour are required for structural engineering risk and reliability estimation. They are essential for the prediction of how the risk of failure can develop with time. Increasingly maintenance of infrastructure is a critical issue but the costs involved are high. Optimal scheduling of maintenance activities increasingly is seen as a sound approach, best achieved through methods based on probabilistic risk-based decision-making of which the theory of structural reliability is a crucial important part [43,44]. Fundamentally the concepts involved are not new but with the development of the powerful methods for estimation of structural system failure probability and the gradual acceptance of Bayesian up-dating techniques [3], the means to carry them out in practice is now possible. This has created, in turn, a need for appropriate and accurate models for structural deterioration. Corrosion models such as outlined herein are attempts to provide such information.

It is clear from the exposition given above, even without the use of direct comparative studies, that linear models and models of the type $A \cdot t^B$ are unlikely to be useful for realistic representation of corrosion losses and for pitting corrosion, except perhaps for short periods of exposure. As has been demonstrated already [34,45] the more refined, multi-phase models reviewed herein are able to throw new light on observations for the effects of alloys on the general and the pitting corrosion of low alloy steels and on the effect of oxygen concentration [26,28] and of nutrient pollution. The simpler models have been unable to do this and have lead to confusion and poor outcomes for correlation studies [46].

Evidently, the models are not complete. In some cases only the mean value is available at the present time. More extensive field studies are required to elucidate information about uncertainty so that probabilistic models can be developed from such understanding. In other cases the effect of environmental factors such as temperature has not yet been investigated. This applies to the influence of water velocity and nutrients. The season of first exposure is a related issue [35]. Of course, the effect of cathodic protection [47] and of protective coatings forms an important input, not considered herein, but capable of research along the lines indicated herein.

For the development of probabilistic models, at the present time the first priority must be to develop better mean-value models for the seemingly most elementary cases, such as tidal, splash and coastal atmospheric corrosion. These models must be based on a sound approach to modelling the physical, chemical, biological and environmental factors involved as well as the understanding of corrosion processes available from both corrosion science principles and empirical observations.

Logically, the development of the probabilistic corrosion models must follow the development of deterministic, bio-physio-chemical corrosion models. These should reflect sufficient physical reality if they are to have predictive power suitable for practical application but they should not be excessively complex. They should identify the key factors involved. The required models might be viewed as 'engineering' models rather than scientific ones, conveying the essential essence of behaviour if not its complete details. This is considered to be an exciting and demanding challenge.

Although the above discussion is confined largely to corrosion as an independent deterioration mechanism, in practice corrosion is not an independent issue for risk and remaining life assessments. Corrosion interacts with applied stresses, fatigue, mechanical damage and, most importantly, with protective systems such as cathodic protection, paint coatings and management practices. In practice these interactions cannot be ignored, even though the actual interactions are not in all cases fully understood. Evidently, these interactions for particular applications also provide a rich field for further research in which probabilistic models will be important.

8. Conclusion

Structural reliability theory necessary for the assessment of the risks associated with corroding infrastructure was outlined herein and it was indicated that for these methods to deal properly with corroding structures, appropriate probabilistic models are required to describe the loss of material due to corrosion or the depth of extreme depth pits.

Recently developed models for general corrosion as a function of time were reviewed. It was shown that environmental influences such as water velocity and nutrients in the seawater can have a significant influence on the corrosion loss. The quantitative relationships for water velocity need to be extended to deal with a range of water temperatures. Similarly, models are required for the effect of nutrient pollution and corrosion loss for a variety of water temperature conditions. As indicated, these models will need to be probabilistic in order to describe phenomena for which current understanding is limited. Similar relationships are desirable for extreme depth pitting of structural steels. For these the effect of DO also requires investigation.

A simple example of a pipeline subject to pitting corrosion showed the dramatic increase in estimated failure probability as the pipeline ages and pitting corrosion increases. It was found that modelling of the longer-term behaviour of pit depth growth is particularly important for probability estimates. As shown in the model for pit depth growth, long term pitting behaviour appears to be associated with anaerobic conditions and is therefore not well represented by models of the type $A \cdot t^B$ which commonly are fitted to short-term observations. This observation suggests that for realistic risk assessments the models derived from much conventional pitting research based on short-term laboratory observations are of limited usefulness.

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