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Probabilistic remaining life estimation for deteriorating steel marine infrastructure under global warming and nutrient pollution

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Abstract:

The longer-term serviceability and structural safety of steel infrastructure exposed to seawater conditions may be affected by global warming and by seawater nutrient pollution. These may affect abiotic and biotic (microbial) corrosion. A model for long-term corrosion is developed from data obtained from steel piling exposed for 33 years in a seawater harbour. The effects on corrosion losses on the structural reliability of steel sheet piling as used in harbours world-wide were investigated as a function of seawater temperature rise from global warming and of seawater nutrient pollution. The results show that structural reliability is more sensitive to likely nutrient pollution than to predicted increases in seawater temperature, noting also that global warming also could increase nutrient pollution from anthropological sources.

Keywords: Structural steel, Corrosion, Climate change, Reliability, Modelling.

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Introduction

The estimation of the long-term structural reliability and remaining safe life is important for items of major structural steel infrastructure subject to aggressive corrosion conditions [1]. Many steel infrastructure assets are provided with protective measures such as paint coatings or sacrificial or impressed current cathodic protection, but these are not always effective, commercially feasible or economic in the long-term [1]. Visual inspection is a commonly used technique for in-situ condition assessment but is feasible only for those parts of the steel structure that are sufficiently accessible for inspection and that can be sufficiently exposed. Such inspection, aided by well-established techniques for in-situ inspection such as ultrasonic testing, is, however, limited in the practical information it can provide and this must be considered in any assessment of reliability and potential future remaining life. Inspection techniques and the various tools available also have associated with them a considerable degree of measurement uncertainty, in part as a result of operator error and in part because of the interpretation required for the results obtained [2]. Taken together, this implies that any estimate of expected life of physical infrastructure will need to consider such uncertainties and thus should be based on (well-established) probabilistic risk or structural reliability analysis [3].

In parallel to the more conventional definition of risk, corrosion risk can be expressed as the product of the probability of failure resulting from corrosion and the consequences from such corrosion [3]. The probability of failure for a given corrosion risk can be estimated on the basis of the severity and the extent of corrosion damage expected to occur on a component. The likely consequence(s) should failure occur can be estimated from the impact of such failure, evaluated against a number of criteria. These include personnel safety, environmental and operational impacts. Once these assessments have been made, the overall corrosion risk assessment then ranks the equipment or the infrastructure according to the corrosion related risks involved. This may then lead to identification of mitigation and management options, following conventional practice [4].

The tools and techniques for such probabilistic risk assessments are well-developed and readily available in the literature [3,4,5].

The present paper focuses on the practical problem of the long-term reliability of steel piling such as used extensively in commercial and other harbours. Fig. 1a shows a simple example after failure. Such sheet piling is known to be prone to a phenomenon known as accelerated low water corrosion (ALWC) [6]. Recently ALWC [7] and also immersion corrosion [8] has been shown to be influenced strongly by both water temperature and anthropological and other forms of water pollution. As reported earlier for reinforced concrete infrastructure, increased water temperatures as a result of expected global warming [9,10] also may have an important influence on the long-term reliability of steel (and other) infrastructure. Also, water pollution largely as a result of anthropological influences has been observed in harbours and coastal regions in many parts of the world, including in regions of high economic activity and generally well-regulated industrial, chemical and agricultural practices [19]. The effect of these influences is considered herein using as a typical application low alloy steel sheet piling such as widely used for quays and retaining systems in seawater harbours.

The next section outlines the development of a simple linear model to represent long-term corrosion loss. Based on observations of the corrosion of similar piling of steel infrastructure directly exposed to seawater conditions this paper reports data and statistical interpretations of data or loss of cross-section loss of mild steel piling exposed for some 33 years in 20°C (aver.) Pacific Ocean seawaters in Newcastle harbour, Australia. The parameters of the proposed model are treated as random variables. A probabilistic analysis of the steel sheet piling is then described, using as input random variables to describe the material and loading characteristics. The effects of feasible changes in water temperature and in water quality are then considered. The results are discussed in the context of the effect of climate change and consequent elevation of average seawater temperature on long-term reliability, safety and serviceability.

(a)

(b)

Figure 1. (a) Failure example of a sheet pile retaining wall (photo courtesy R Jeffrey), (b) “Bright orange” localized corrosion of a tubular pile showing local wall perforation just above the water line at extreme low tide [17].

Marine corrosion of steel

Information and data for the long-term corrosion of steel in marine conditions largely is confined in the literature to exposures much less than 10 years. The few data for up to 16 years exposure [11,12,13,14] indicate that the long-term trend in uniform corrosion $c(t)$ and also in maximum pit depth tends to follow an anaerobic steady state [11] linear trend as a function of time t that can be described, for exposure periods greater than about 5 years by Eqn. 1:

$$c(t) = c_s + r_s \cdot t, \quad t > 5 \text{ years} \quad (1)$$

where c_s is the ‘y-intercept’ at time $t = 0$ and r_s is the slope of the long-term trend. For the present study data for both immersion corrosion and for the ALWC zone became available as a result of renovation of steel sheet piling in Newcastle harbour, Australia. This harbour is known to have relatively low levels of water pollution owing to high tidal flushing [20]. Typically, steel sheet piling in older harbours are not protected since the historical corrosion losses were considered to be acceptably low [19]. However, in 2008 ‘bright-orange’ localized corrosion of steel sheet piling was observed just below low water level and within the lower half of the tidal zone of some steel piles. These showed high cross-section losses or even complete wall perforations (Fig. 1b). At the time

both microbiologically induced corrosion (MIC) and Accelerated Low Water Corrosion (ALWC) were suspected as contributing to the high corrosion losses observed [9,19,20] and this has since been shown to be consistent with observations elsewhere [7,8]. Port Corporation records showed that the piles were around 33 years old and had not been given protective coatings when first installed. Replacement of the affected piles permitted measurement of their corrosion losses to obtain estimates of the parameters c_s and r_s (Eqn. 1) both for long-term ALWC and for long-term immersion corrosion, as described below.

The piles showing high corrosion losses were withdrawn from service. Four of these the parts, exposed in the tidal zone and in the upper part of the immersion zone, were made available to the present study (Fig. 2a). The locations of high and low tide levels were marked (Fig. 2b), identified by barnacle growth. The four piles also were arranged so as to correlate the tidal range zone. An electric jack-hammer was then used to remove dried barnacles as well as to remove loose rust, leaving only the harder denser interior rusts (Fig. 2b). Each region was examined in particular for the location of complete wall perforations. Samples (250 x 250 mm in size) were cut from the tidal and the ALWC zones for measurement of localised corrosion and pit depth relative to the surrounding metal in a manner similar to that previously reported for weld zones [18]. In addition, ultrasonic testing was used to take two independent set of measurements (*i.e.* A and B) of the remaining wall thickness of the parent metal every 150 mm in the longitudinal direction. It is then reasonable to assume that the readings are statistically independent as they were taken in longitudinal strips along the piles equally distant and away from the two longitudinal welds, one on each side of each tubular pile. These two sets of readings are labelled 'A' and 'B'. The results are shown in Figs. 3-6.

(a) (b)

Figure 2. (a) Steel piling samples as received showing tidal region: marked by excessive barnacle growth in the tidal zone and (b) cleaned piles arranged by location of the tidal range with high tide locations marked (at right). The tops of the piles were at the right-hand side of these samples.

In Figs. 3-6 the grey area represents profile of the remaining wall thickness along the line of ultrasonic measurements as constructed from the readings at each point. Each of individual thickness measurement shown on Figs. 3-6 is the average of 5 individual spot readings over an area of 625 mm², consistent with relevant standards [19]. It is clear that the profile for pile 3 differs significantly from piles 1, 2 and 4. Such difference can be seen also from the overall view of the piles in Fig. 2b. It indicates that not all piles at a given general location necessarily corrode in the same manner. This has been noted also for steel sheet piling [6]. For the present purposes pile 3 was ignored in the analysis as unrepresentative of the likely worst scenario for pile condition, noting that the profiles for the other 3 piles are very similar.

Figure 3. Longitudinal ultrasonic thickness measures for Pile 1. For each pile two sets of longitudinal thickness readings were taken, shown as 'A' and 'B'.

Figure 4. Longitudinal ultrasonic thickness measures for Pile 2. For each pile two sets of longitudinal thickness readings were taken, shown as 'A' and 'B'.

Figure 5. Longitudinal ultrasonic thickness measures for Pile 3. For each pile two sets of longitudinal thickness readings were taken, shown as 'A' and 'B'.

Figure 6. Longitudinal ultrasonic thickness measures for Pile 4. For each pile two sets of longitudinal thickness readings were taken, shown as 'A' and 'B'.

Statistics of corrosion losses

The original wall thickness was nominally 16mm [17]. This was verified by careful examination of the samples for regions that appeared not to have corroded or to have corroded very little. Nevertheless, some uncertainty remains about the original wall thickness and its possible variation between the samples and over each sample surface. For all piles other than pile 3 the corrosion loss was determined from the difference between the remaining wall thickness and the nominal original thickness.

Consistent with the visual observations of the patterns of corrosion for each pile, the data collected from the longitudinal wall thickness measurements and the resulting corrosion losses were divided into sub-populations based on the zones defined by tide levels, namely, from the top (right of Fig. 7) as splash zone, tidal zone (inter-tidal range), ALWC zone and immersion zone. The representative distances are shown on Fig. 7. The origin was chosen arbitrarily when the piles were in corresponding positions (Fig. 2b).

The resulting wall thickness losses were then analysed using standard statistical techniques to obtain the statistics for each zone as shown in Table 1. It is immediately evident that the mean loss and its standard deviation for the ALWC zone are much greater than the mean losses and standard deviations for the other zones. Similar results have been reported for comparable exposure conditions [7].

Figure 7. Steel piling subdivided into four data regions based on tidal levels.

Table 1. Statistics for corrosion loss of 33 year-old exposed steel piling regions (mm).

The most appropriate probability distribution to represent the data for corrosion loss in each zone also was considered. Several were tried. Table 2 shows the best-fit distribution for each zone, and the values of several fitting criteria. For the ALWC region the log-normal distribution is an appropriate probability distribution function. For the others a Frechet probability function is the best fit.

Table 2. Best fit analysis for the data for each exposure region (Fig. 7) at 33 years of exposure.

The statistical results at 33 years of exposure can be used in Eqn. 1 to determine the parameters c_s and r_s and the linear trend representing long-term corrosion. The estimated corrosion loss values are shown in Fig. 8 for corrosion in the ALWC one and in the tidal zone. Also, information about c_s was taken from [8] to give values of expected losses at close to time $t = 0$. From these the means were used to estimate the trends for the ALWC and the tidal zones. Trends for the splash zone and the immersion zone are similar but are not shown. Also shown for illustration is the effect on the trends as a result of a 6°C rise in average seawater temperature. The rationale for such a change is considered in the next section.

Figure 8. Interpreted long-term corrosion loss trends for the ALWC zone and the tidal zone, based on the results shown in Tables 1 and 2 and in Ref [8]. Also shown are the effects of a 6°C rise in average water temperature.

Effect of temperature increase due to climate change

It has been proposed as likely that there will be a rise in average seawater surface temperature of up to 6°C over the next 100 years [20]. Although this is likely to be a slow increase over time, it does provide a basis for considering its effect on corrosion losses. The effect of temperature on the parameters c_s and r_s has been determined previously for relatively shorter-term observations [8,21]. However, since no other information about the long-term effect of temperature on corrosion is available, those relationships were assumed valid also for long-term observations and effects.

Effect of nutrient pollution

The long-term trends for the corrosion of steels in seawater have been shown to be a function of water quality and in particular the concentration of dissolved inorganic nitrogen (DIN) [8], a critical nutrient for microbiological activity. The severity of ALWC also has been correlated with DIN [7]. To obtain some estimate of potential rises in DIN over a longer period of time, reported nutrient levels for the North Sea [15] may be considered indicative. These show that average DIN concentrations of 0.2-0.6 mgN/L have been reported for surface waters for winter and summer conditions [15]. It appears that regardless of seasonal effect, increases in land based sources account for the majority (65-80%) of total nitrogen inputs [15]. The effect on corrosion is that the introduction of large quantities of nutrients can lead to a considerable increase in bacterial and algal biomass. This can lead to localized anaerobic conditions and unfavourable corrosion conditions [21] as also seen in empirical in-situ studies [7,8].

Example

Consider now, as an example, a newly installed sheet piling retaining wall, such as that shown in Fig. 9, designed and expected to be serviceable for 100 years. Let it be assumed, as is reasonable in practice, that the piles are unprotected, having no protective paint coatings or cathodic protection. Let it be assumed also that failure of the retaining wall will halt all dock works and associated services. Failure may be defined as excessive deformation of the wall such as by visually noticeable deformation of pavements and dock areas. This failure takes place when the stress reaches a critical stress state for structural failure and can be defined as the onset of yield. Prior to this elastic behaviour governs. Thus yield is considered for capacity and can be assessed by plastic hinge initiation in the structural piles.

A sheet pile wall (Fig. 1a) forms a structural system of parallel steel units. Because of their connectivity, they can be expected, to a large degree, to behave very similarly under applied loading. Also, despite some observations of differences in corrosion behaviour between even piles closely adjacent [6] as seen also in Fig. 5 for pile 3, overall it can be expected that corrosion losses will be generally similar. For the purposes of sheet pile analysis, and for the reliability analysis, it will be reasonable, if slightly conservative, to assume reaching of the failure condition for a single pile is equivalent to failure of the whole system. There is empirical evidence for this in observed failure cases [4] and as in Fig. 1a.

It is accepted practice to model piling for structural analysis as an embedded retaining wall [18]. Thus the assessment of soil stratification and the assignment of appropriate engineering parameters are important. In the present case it is assumed also that the effects of ALWC are the most important for structural failure and is the single most important cause a localized loss of cross

section. This is consistent with field observations [4]. This means the pile wall will behave as a cantilever, embedded in the soil but with yield occurring in bending within the ALWC region (Fig. 9). For simplicity of analysis, it will be assumed that the point marked X in Fig. 9 represents the location where bending failure is assumed to occur.

Figure 9. Cross-section of a steel pile retaining wall, showing applied loadings, soil support conditions and the resulting structural active and passive soil and other pressures similar to those typically assumed in design. ALWC represents the location of mean location of the ALWC zone where failure by cantilever action is assumed to occur.

Also in Fig. 9, *MHT* is the median high tide, *MT* is mid tide, *MLT* is median low tide, h_{1-4} is height of each layer of soil, h_5 is the variable water tide level, *X* represents the location of the mean of the ALWC zone, *Q* is the surface surcharge live loading, *GWL* is the groundwater bed level, γ_s is the bulk weight density of loose fine sand, ϕ'_s is the effective stress angle of shear resistance for each soil layer, γ_{sat} is the saturated mass density of loose fine sand, γ_w is the density of water, γ_c is the bulk density of compacted fine sand, γ_g is the bulk density of sand and gravel, A_{1-4} is the active destabilising stress loading due to water bed level and various soil layers, and A_5 is the stabilising passive stress loading varying according to tide levels.

The overall stability of cantilever retaining walls depends solely on the passive stabilising pressure to withstand soil and/or water active destabilising loads (Fig. 9). These pressures must be estimated and thus there can be considerable uncertainty as well as localised variations in soil properties. In addition, retaining wall height and water conditions along a wall can have significant effects on the alignment of a cantilever wall and thus could be considered as random variables.

To be specific, consider a practical application using the U profile sheet pile. Let this be AU 25. Its dimensions, width (*b*), height (*h*), thickness (*d*-top flange, *s*-angled flange) and other properties are summarized in Table 3.

Table 3. AU 25 commercial steel sheet piling nominal dimensions (mm) [22].

To estimate the probability of failure p_f of the steel piling, let the limit state function be defined as usual as:

$$p_f = P[G(t) < 0] \quad (2)$$

where $G(t)$ is the time dependent limit state function, defined as the difference between the steel pile yield stress capacity (f_y) and maximum stress on the critical pile cross-section (σ_{max}) caused by the applied loading, or full penetration of pile cross-section due to corrosion as respectively per:

$$G(t) = f_y - \sigma_{max}(t) \quad \text{or} \quad G(t) = s - c(t) \quad (3)$$

As usual, $G(t) > 0$ indicates the safe region, $G(t) = 0$ indicates the failure surface and $G(t) < 0$ indicates the failure region. For bending about the point *X*, the bending stress in the steel pile wall is given by:

$$\sigma_{max}(t) = (M_{max}(t) \cdot y) / I \quad (4)$$

where $\sigma_{max}(t)$ is the maximum normal stress as a function of time, acting on a cross-section, *y* is the distance from the centroidal axis to that point, caused by the local bending moment $M_{max}(t)$ which

is, in general, a function of time, and I is the second moment of area, about the centroidal axis for the pile cross-section, typically uniform throughout the pile length. The moment $M_{max}(t)$ may be evaluated from the areas of the active and the passive stress diagrams and their respective lever arms about the point X as follows:

$$M_{max}(t) = \sum_{i=1}^4 (A_i \cdot l_i) - (A_5 \cdot l_5) \quad (5)$$

The random variables and their means and coefficients of variation (COV) adopted for the present example are summarized in Table 4. These values are representative and reasonable values, based on the specified pile section and on typical soil properties [22].

Table 4. Basic design random variables and statistical properties assumed.

The uncertainty associated with the parameters c_s and r_s for the corrosion losses (Eqn. 1) is an important factor in the reliability analysis. As no published information is available about the mean and COV of parameter c_s they were obtained from previous work [8] for Newcastle harbour. Both are functions of average annual seawater temperature and DIN. Parameter r_s mean was obtained from Eqn. 1 by considering the slope intercept (c_s) and average corrosion loss given from the ALWC 33 year data ($r_s = (8.4 \text{ mm} - 0.502 \text{ mm}) / 33 \text{ years} = 0.239 \text{ mm/year}$). A COV of 0.58 for r_s is taken from ALWC 33 data statistic as representable of long-term slope variability. To consider the effects of water pollution and water temperature, increases in DIN and average water temperature were applied, using as increased DIN 0.4 mgN/L (a typical change based on North sea experience [15]) and as temperature increase 6°C over 100 years [20]. The *temp. adj.* r_s and the *nutri. adj.* r_s were recalibrated based on previous reports [8,21] which plots slope parameter r_s as a function of average annual seawater temperature and DIN along with ratio of ALWC/immersion corrosion loss. A 6°C temperature increase would increase corrosion loss roughly by 0.015mm/year ($r_s = (8.4\text{mm} + 0.015\text{mm} \cdot 33 \text{ years} - 0.502 \text{ mm}) / 33 \text{ years} = 0.254 \text{ mm/year}$) and an added 0.4 mgN/L of DIN would increase slope r_s by 0.03 mm/year. For both applied at the same time the slope parameter r_s is adjusted by +0.045 mm/y. For purpose of illustration tide levels at Newcastle Port are used herein. According to the Newcastle Port 2014 tide chart and information [19], tide levels average range is 1.5m, varying about 1m between summer and winter seasons. Considering the worst case scenario for design purposes, h_5 presents a mean of 1.5m and COV of 0.67. The results are summarized in Table 4.

The reliability analysis was performed using the Monte Carlo reliability method [5] with 5 million runs. For the effect of added DIN and water temperature rise, the numerical analysis was performed by considering a linear time-dependant increase of temperature of 6°C over a period of 100 years as well as taking a linear readjustment with time of the average values of slope gradient r_s as per their empirically established relationship [8]. The results obtained are discussed in the next section.

Results

For the above example with the data in Table 4 the structural system reliability analysis showed that with increased exposure time the failure probability increased as shown in Fig. 10.

Figure 10. Pile probability of failure for given example and estimations considering effects of seawater warming, elevated DIN nutrients and combined temperature and nutrient effect. Failure estimates for the tidal region is also shown for comparison.

By considering a reliability index of 2.3 ($P_f = 0.01$) and the corresponding probability of failure as acceptable for Australian harbour structures [24] the probability of failure for the ALWC case reaches the limit of acceptable probability for design risk (collapse) after only 14 years of exposure. Steel piles exposed in Tidal waters (*i.e.* using the Tidal zone data) would reach this threshold after 31 years of service. Using the adjusted ALWC data sets, the structural reliability estimates show that there is a difference in asset life serviceability estimations, with the estimated probability of failure reaching the threshold after 13 years for Temperature adjusted data; 12 years for nutrient adjusted data; and 11 years for both temperature and nutrient adjusted data. Overall, these results are consistent with practical failure reports, which show that some piling structures are deemed unsafe 10 to 20 years earlier than its design life [21].

For all cases wall perforation was the dominant failure as the stress states were well below the permissible yield stress. This is expected for accelerated corrosion, as illustrated in Fig 10 where there is a rapid increase in the probability of failure as a function of time for all data cases only after a few years into service. This result most likely is due to influence of microbiological corrosion, known to be a factor in this exposure zone [8,21] in which microbiological growth tends to occur exponentially under elevated nutrient conditions. This means that it could be expected that risk levels would increase rapidly with time. However, what is perhaps alarming is the difference in the rate of increase in probability of failure for the ALWC data and compared with the rate of probability increase for the Tidal data. Indirectly this also shows the importance of correct interpretation of the phenomena that influence corrosion processes. A good understanding of the factors in corrosion mechanics is important for accurate estimation and prediction [3].

Overall, the trends for the probability of failure with temperature and nutrient increase (Fig. 10) are similar to the ALWC trend. The most noticeable difference is that risk levels increase at an earlier stage. As expected, a slow increase of water temperatures with time would produce a somewhat constant effect on slope r_s of the long-term corrosion loss. As noted, Fig. 10 shows that the probability of failure increases with increasing service life t , as would be expected. It is seen in Fig. 10 that the probability of failure increases very considerably after about 10 years of exposure for the ALWC data and the adjusted data, while Tidal data presents a lower increase of failure probability with time.

A sensitivity analysis was carried out to assess the relative contributions (α^2) and the associated probabilities of failure (P_f) as a function of service life, t (exposure period in years). This was done by changing the mean of each random variable by 10% and observing the effect of such change on the probability of failure. This is a well-established technique [10, 25]. The relative contribution of some of the variables (*e.g.* tide level, soil parameters, cross section dimensions) is very low and this continues to be the case over the total service life. This indicates that these random variables contribute little to the overall failure probability. Similarly, steel yield stress (f_y), passive pressure tidal height (H_5) and top flange thickness random variable (d) were found to have only a small influence on the overall reliability estimates. However, their contributions were found to decrease gradually with increased service life. The relative contribution of the slope gradient (r_s) and flange thickness (s) are lightly greater. Overall, slope parameter (r_s) is clearly the variable which has strongest effect on reliability estimations.

The statistics given in Table 2 are for steel that has been exposed for 33 years to seawater. This means that the corrosion loss process was well into the long-term corrosion range. As has been shown by extensive data analysis and model studies [1,9,16,20,21] this means the corrosion processes and hence the corrosion behaviour can be considered to be in a different phase to the corrosion processes that apply much earlier. It follows that, as represented by the bi-modal model for the progression of corrosion [8], the population from which data might be extracted, changes with the progression of time. Until recently a lack of suitable long-term data resulted in an

insufficient understanding of the changes in the corrosion processes over a long period of time and the related fundamentals involved often led to the erroneous assumption that all data for corrosion, no matter how long the exposure period, arise from a single population of measurements [cf. 26, 27,28]. It follows that for the prediction of long-term corrosion and associated probabilities of occurrence, statistics for long-term corrosion are of interest. These are shown in Table 2.

Overall the above results and observations are consistent across all three data sets analysed: ALWC, temperature adjusted ALWC, nutrient adjusted ALWC and combined temperature and nutrient adjusted ALWC. They confirm the expectation that long term corrosion parameter r_s plays an important role in the reliability of marine infrastructure such as steel piling that typically have relatively long service lives. This expectation is reflected also in the sensitivity analysis, where r_s is shown to have a much stronger influence than the other parameters. It is considered the reason for the sudden increase in failure probability seen in Figs. 10 after a certain period of exposure (10 or more years). However, the outstanding difference between the results is the difference in the probability of failure estimated using the ALWC data compared with the Tidal zone data for the uncertainty related to cross sectional loss (*i.e.* it is the form of corrosion that mostly likely leads to failure). This is clearly illustrated by the trends in Fig. 10.

As noted, ALWC is a function both of annual mean seawater temperature and microbiologically influenced corrosion [8,21,22]. The latter also is influenced by seawater temperature. High corrosion losses of steel piling due to ALWC have been reported [7,17,21,22] immediately below the low tide compared with corrosion losses in the tidal zone. In addition, environmental changes such as a long term increase in seawater temperature can have an effect on the amount of nutrients available for bacterial metabolism, largely because of land-based changes causing greater nutrient run-off [18]. This has a direct effect since both c_s and r_s are positively correlated with annual average concentration of dissolved inorganic nitrogen (DIN) in local bulk seawater [7]. If it is assumed as an example, and conservatively where no pre-emptive mitigation action was taken, that there is a seawater temperature rise approximately equal to a climate change of 6°C, it is likely that the increase in water temperature will have a slow but very considerable effect on marine fauna and flora. In turn this likely to have a major influence on corrosion through microbiological effects since, apart from most bacteria, not all species are likely to survive such changes and dead organic material constituents a source for bacterial metabolism [25].

Conclusions

Marine corrosion of steel infrastructure, including piles, is known to be a function of average seawater temperature and the local concentration of nutrients necessary for microbiologically influenced corrosion. A linear, long-term corrosion loss model was calibrated to data obtained from the 33 year service corrosion of steel tubular piles in Newcastle Harbour and the variability in the model parameters estimated. Using accepted models for global warming the rate of increase in seawater temperature and its variability was estimated. The reliability analysis for typical sheet piling exposed to nutrient pollution similar to that of the North Sea and to predicted increased seawater temperature showed that the rate of long-term corrosion loss has a variability about one order of magnitude greater than the variability of all other parameters, not considering the uncertainty in prediction of climate change or increase in seawater temperature. Variability of the long-term corrosion rate therefore has a major influence on the overall probability of failure estimates. The results also show that structural reliability is more sensitive to nutrient pollution than to predicted increases in seawater temperature, noting also that nutrient pollution from anthropological sources might be increased by global warming.

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Table 1. Statistics for corrosion loss of 33 year-old exposed steel piling regions (mm).

Region	Sample size	Range	Mean	Std. Dev.	COV	Min	Max
Immersed	16	8.9	2.2	2.5	1.14	0.6	9.5
ALWC	32	14.6	8.4	4.8	0.58	1.4	16.0
Tidal	34	7.9	2.6	1.9	0.73	1.0	8.9
Splash	46	3.4	2.1	0.8	0.40	0.7	4.1

Table 2. Best fit analysis for the data for each exposure region (Fig. 7) at 33 years of exposure.

	Immersion Region	ALWC Region	Tidal Region	Splash Region
Fit Distribution	Frechet	Log-Normal	Frechet	Frechet
Parameters	$\alpha = 1.6335$; $\beta = 0.97512$	$\sigma = 0.8237$ $\mu = 1.8613$	$\alpha = 2.2987$; $\beta = 1.6679$	$\alpha = 2.8737$; $\beta = 1.5766$
Kolmogorov Smirnov	0.13	0.20	0.09	0.15
Anderson Darling	0.43	2.03	0.33	0.91
Chi-Squared	0.12	7.34	2.01	5.53

Table 3. AU 25 commercial steel sheet piling nominal dimensions (mm) [22].

Product Section	Width (b)	Height (h)	Top Flange Thickness (d)	Angled Flange Thickness (s)	Sectional Area	Single Pile Mass	Moment of Inertia	Static Moment
	mm	mm	mm	mm	cm ² /m	Kg/m	cm ⁴ /m	cm ³ /m
AU 25	750	225	14.5	10.2	188	110.4	56240	1420

Table 4. Basic design random variables and statistical properties assumed.

Symbol	Unit	Type	Mean	COV	Symbol	Unit	Type	Mean	COV
f_y	N/mm ²	Lognormal	275	0.07	d	mm	Normal	14.5	0.05
h_1	m	--	0.4	--	s	mm	Normal	10.2	0.06
h_2	m	--	0.2	--	h	mm	Normal	225.0	0.02
h_3	m	--	1.4	--	b	mm	Normal	750.0	0.02
h_4	m	--	0.2	--	c_s (ALWC)*	mm	Normal	0.502	0.07
h_5	m	Normal	1.5	0.67	r_s (ALWC)	mm/year	Lognormal	0.239	0.58
Q	kN/m ²	Gumbel	2.931	1.05	r_s (ALWC) <i>temp adj.</i>	mm/year	Lognormal	0.254	0.58
γ_s	kN/m ³	Normal	14.7	0.02	r_s (ALWC) <i>nutri adj.</i>	mm/year	Lognormal	0.269	0.58
γ_{sat}	kN/m ³	Normal	19.1	0.02	r_s (ALWC) <i>temp+nutri adj.</i>	mm/year	Lognormal	0.284	0.58
γ_c	kN/m ³	Normal	19.4	0.02					
γ_w	kN/m ³	--	9.81	--					
γ_g	kN/m ³	Normal	20.6	0.02					

* c_s is maintained for all adjusted linear extrapolations (*temp. Adj.*; *nutri. adj.*; *temp.+nutri. adj.*) as recalibration differences are negligible.

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Figure 1. (a) Failure example of a sheet pile retaining wall (photo courtesy R Jeffrey), (b) “Bright orange” localized corrosion of a tubular pile showing local wall perforation just above the water line at extreme low tide [17].

Figure 2. (a) Steel piling samples as received showing tidal region marked by excessive barnacle growth in the tidal zone and (b) cleaned piles arranged by location of the tidal range with high tide locations marked (at right). The tops of the piles were at the right-hand side of these samples.

Figure 3. Longitudinal ultrasonic thickness measures for Pile 1. For each pile two sets of longitudinal thickness readings were taken, shown as ‘A’ and ‘B’.

Figure 4. Longitudinal ultrasonic thickness measures for Pile 2. For each pile two sets of longitudinal thickness readings were taken, shown as 'A' and 'B'.

Figure 5. Longitudinal ultrasonic thickness measures for Pile 3. For each pile two sets of longitudinal thickness readings were taken, shown as 'A' and 'B'.

Figure 6. Longitudinal ultrasonic thickness measures for Pile 4. For each pile two sets of longitudinal thickness readings were taken, shown as 'A' and 'B'.

Figure 7. Steel piling subdivided into four data regions based on tidal levels.

Figure 8. Interpreted long-term corrosion loss trends for the ALWC zone and the tidal zone, based on the results shown in Tables 1 and 2 and in Ref [8]. Also shown are the effects of a 6°C rise in average water temperature.

Figure 9. Cross-section of a steel pile retaining wall, showing applied loadings, soil support conditions and the resulting structural active and passive soil and other pressures similar to those typically assumed in design. ALWC represents the location of mean location of the ALWC zone where failure by cantilever action is assumed to occur.

Figure 10. Pile probability of failure for given example and estimations considering effects of +6°C seawater warming, +0.4 mgN/L elevated DIN nutrients and combined temperature and nutrient effect. Failure estimates for the tidal region is also shown for comparison.