Spatial Variability of Pitting Corrosion and Its Effect on the Strength and Reliability of Prestressed Concrete Bridge Beams

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SYNOPSIS

Corrosion of reinforcing and prestressing steel due to chloride attack is one of the major causes of deterioration of concrete structures. This is of most concern because of the associated reduction in steel cross-sectional area, spalling and loss of bond. In the case of prestressed concrete structures, the corrosion of prestressing strands can lead to a sudden failure due to higher stress levels in the steel.

The paper deals with the development of probabilistic models to predict the strength and reliability of prestressing strands subjected to pitting corrosion. A pitting corrosion model was developed from accelerated corrosion tests in a chloride-concrete environment. From the accelerated corrosion tests, the spatial distribution of maximum pit-depth along strands for various lengths and corrosion rates is developed. From the model, the section loss of the strand can then be calculated. The probabilistic model can also be combined with appropriate failure criterion to calculate the probability of failure of prestressing strands under pitting corrosion attack.

Finally, the probabilistic model of pitting corrosion is combined with Finite Element Analysis and models of corrosion initiation and propagation to study the effect of pitting corrosion on prestressed concrete bridge beams. This will allow time to failure and estimates of structural reliability to be calculated. From the analysis, it was found that the corrosion rate and the failure criterion have a significant effect to the time to failure for a girder. This probabilistic approach will lead to more realistic predictions of the actual behaviour of prestressed concrete bridge beams suffering corrosion attack.

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1 INTRODUCTION

Corrosion of reinforcing and prestressing steel due to chloride contamination is one of the primary causes of deterioration of reinforced concrete structures. In general, corrosion is of most concern because of the associated reduction in steel cross-sectional area, spalling and loss of bond, which with time will lead to loss of strength and serviceability. In the case of prestressed concrete structures, the corrosion of prestressing strands may trigger the collapse of the structures due to higher stress levels in the steel. The sudden collapse of The Saint Stefano bridge in Italy on March 1990 as reported by Proverbio and Ricciardi (12) and pedestrian bridges at Lowe's Motor Speedway in North Carolina (Goin (9)), clearly highlight the importance of the problem.

Two types of corrosion phenomena that may affect prestressing steel are general (uniform) and pitting (localized) corrosion. General corrosion can be found when there is a relatively uniform surface attack over the perimeter of the steel. This mainly occurs as a result of reaction between the hardened cement paste with carbon dioxide. This process, commonly known as carbonation, can lower the pH of the concrete to around 8, destroying the passivity and allow the corrosion to initiate. In contrast, pitting corrosion arises especially in places where the metal protective layer (passivation) has been damaged locally by aggressive ions (e.g. chlorides). Chlorides may come from variety of sources, such as salt spray in marine environments, from the application of de-icing salts during winter in cold region and also in some cases from concrete admixtures. Pitting corrosion is of more concern than general corrosion since the loss of cross-sectional area can be quite high in localised areas along a prestressing strand or reinforcing bar.

Pitting corrosion varies spatially along the length of the reinforcing bar. However, majority of studies have so far only modelled the strength and reliability of flexural members subject to deterioration by concentrating on regions of peak actions (i.e. mid-span capacities) or other "critical" sections. Therefore, ignoring the spatial variability of pitting corrosion will lead to an under-estimation of probability of failures of structures, at least for members in flexure.

The intention of the present paper is to consider the effect of spatial variability of pitting corrosion on structural strength, mean time to failure and structural reliability, for prestressed concrete beams in flexure. The analyses considered a typical PC bridge girder located at a coastal environment. The paper first reviews briefly the development of probabilistic models to predict the spatial distribution of maximum depths of pitting for prestressing strands subjected to pitting corrosion. The pitting corrosion model was developed using accelerated corrosion tests in a chloride-contaminated concrete environment. The probabilistic model of pitting corrosion will then be combined with non-linear Finite Element Model (FEM) and models of corrosion initiation and propagation to study the effect of pitting corrosion on prestressed concrete bridge beams. This will allow time to failure and estimates of structural reliability to be calculated. Including the effect of spatial variability allows for the analysis to consider failure of strands at any location along the beam, as well as progressive failure of multiple strands as failure of one strand will increase the stress in remaining strands thus leading to possible failure of other corrosion-affected strands. This progressive failure of strands will continue until the beam collapses. The beam under consideration is discretised into elements where the maximum pit depth is assumed statistically independent for each element.

For convenience, the present analyses ignore the spatial variation of dimensional and material properties of the structures even though it is recognised that concrete quality and concrete cover do vary spatially over the concrete structures (e.g., Faber & Rostam (7), Vu & Stewart

(18)), usually caused by different concrete batches and the variability of workmanship. These spatial variables, however, will not have as significant an influence on structural reliability of flexural members as pitting corrosion. Improved estimates of structural performance and reliability allow for more realistic predictions of remaining service life, particularly if service life prediction is based on reliability-based safety or life-cycle cost criteria.

2 PROBABILISTIC MODELS OF PITTING CORROSION

The actual mechanism of pitting corrosion is not yet fully understood. However, the use of extreme value theory appears to give promising results when modelling pitting corrosion in aluminum (Aziz (3)) and steel (Eldredge (6), Finley and Toncre (8), Sheikh et al (13)). Vajo et al (15) recently also successfully applied this theory to analyse crevice corrosion under elastomeric seals to aluminum surfaces. Darmawan & Stewart (5) have found that the distribution of maximum pit depths for prestressing wires is best represented by the Gumbel (EV-Type I) distribution.

The geometric model proposed by Val & Melchers (16) is used to predict the loss of cross-sectional area for a pit size of depth a, see Figure 1.



Figure 1: Pit Configuration

2.1 Prestressing Strands

To obtain real data on pitting phenomena, an accelerated corrosion testing regime was carried out at The University of Newcastle. Corrosion rates of $150-420\mu$ A/cm² were introduced in steel wires and strands embedded in chloride-contaminated concrete specimens in order to measure pit depths along 1.5 m lengths of cold-drawn prestressing wires (5.03 mm diameter) and 7-wire strands (12.7 mm diameter). Load tests on the corroded wires and examination of the fracture surface reveal that the mode of failure is yielding with reduced ultimate failure strains, and not stress corrosion cracking or brittle fracture (Darmawan & Stewart (5)).

The predicted Gumbel distribution of maximum pit depth (a) at any time of exposure T (years), corrosion rate i_{corr} ($\mu A/cm^2$) and wire length L is thus

$$f_{a}(T,i_{corr},L) = \frac{\alpha}{\lambda^{0.54}} e^{-\alpha \left(\frac{a}{\lambda^{0.54}}-\mu\right)} e^{-e^{\alpha \left(\frac{a}{\lambda^{0.54}}-\mu\right)}}$$
(1)

where

$$\boldsymbol{\lambda} = \frac{T(2D_{o} - 0.0232i_{corr}T)}{T_{o}(2D_{o} - 0.0232i_{corr}T_{o})} \qquad T_{o} = \left[\frac{i_{corr-exp}}{i_{corr}}\right]T_{o-exp}$$
(2, 3)

$$\mu = \mu_{o} + \frac{1}{\alpha_{o}} \ln \left(\frac{L}{L_{o}} \right) \qquad \alpha = \alpha_{o}$$
(4, 5)

 μ_o and α_o are the parameter of the Gumbel distribution as obtained from the tests (see Table 1), D_o is the initial diameter of the wire, L_o is the length of the wire used to record the maximum pit-depths, T_{o-exp} is the period of the experiment and $i_{corr-exp}$ is the corrosion rate used in the experiment. For a wire in a presressing 7-wire strand, the Gumbel parameters μ_o and α_o were obtained from maximum pit depths recorded for 96 650 mm length strands. See Darmawan & Stewart (4) for full details of the model development.

Table 1: Gumbel parameters for maximum pit-depths for a single wire

T _{o-exp}	i _{corr-exp}	Lo	$\mu_{\rm o}$	$\alpha_{\rm o}$	no. of samples
(years)	$(\mu A/cm^2)$	(mm)			
0.03836	279	650	1.09	7.35	96

It is assumed that the outer six wires of a 7-wire strand corrode independently of each other. This means that there is increased likelihood of greater pit depths in a strand than an individual wire. There is also a redistribution (increase) in wire stress after failure of each successive wire. Figure 2 shows the distribution of time to failure for a corroding wire and strand.



Figure 2: Distributions of Time to Wire and Strand Failure

3 PRESTRESSED CONCRETE (PC) BRIDGE GIRDER

The bridge considered in this study is a typical simple span PC Bridge, which has a span of 21 m and a clear roadway width of 8.4 m. The bridge consists of four precast prestressed AASHTO Type IV girders (see Figure 3) with equal spacing of 2.3 m and a 200 mm thick cast-in-place concrete deck. The girder was designed according to the AASHTO LRFD

Bridge Design Specifications (AASHTO (1)) assuming bonded tendons, unshored construction and no composite action between the girder and the cast-in-place slab.



Figure 3: AASHTO Type IV bridge girder

Three different components of dead loads are considered: precast concrete, cast-in-place deck and 90 mm asphalt overlay. Axle spacings and distribution of axle loads were calculated based on a U.S. HS-20 truck (AASHTO (1)) and these were used to calculate peak flexural actions. The design concrete strength \vec{F}_c used for the girder is 34.5 MPa and the ultimate tensile strength of the prestressing steel is 1750 MPa. Minimum specified cover is 50 mm. A total of 26 7-wire strands (12.7 mm diameter) were required in the middle of the span to carry the total design loads.

4 FINITE ELEMENT MODELLING AND STATISTICAL PARAMETERS

A three dimensional finite element model (FEM) was employed in this study using commercially available software (ABAQUS (2)). The model includes two types of finite element: eight-node solid elements with reduced integration and truss elements. The first element was used to model the concrete element whereas the second one was used to model the prestressing steel. The truss elements were embedded in the concrete element, which means bond-slip effects were not considered in the analysis. This assumption is acceptable as pitting corrosion generates an oxide of iron different from the rust as a result of general corrosion, with lower volume per unit mass. As such, pitting corrosion is less likely to cause the disruption of concrete cover and hence no reduction of bond strength around the pit.

Element length is taken equal to twice the development length of the strand. The element length of the strand was taken as 1 m. Uniaxial elastic-plastic material models were used for both steel and concrete stress strain relationships. For concrete, the linear tension stiffening effect after cracking was included to take into account the effect of interaction between steel and concrete after flexural cracks have formed.

The analysis assumes that the PC bridge girder is located close to the coast and the source of deterioration is atmospheric chlorides (sea-spray). Fick's second law of diffusion is used to predict corrosion initiation and the corrosion rate is assumed constant with time, see Vu &

Stewart (17) for further details. The total loss of prestress is estimated based on AASHTO specifications as only long-term behaviour is considered for this study. A summary of statistical parameters representative of PC bridge girders in the U.S. is given in Table 2.

Parameter	Mean	COV	Distribution
f' _{cyl} (concrete cylinder strength)	$\dot{F_{c}^{a}} + 7.5 \text{ MPa}$	σ=6 MPa	Lognormal
C^{b} (concrete cover)	C_{nom}^{c} +8.6 mm	σ=7.4 mm	Normal
f' _{ct} (concrete tensile strength)	0.59 - f' _c	0.13	Normal
E _c (concrete modulus elastic)	4600 f _c	0.12	Normal
Model error i _{corr} (corrosion rate)	1.0	0.2	Normal
C _o (chloride surface concentration)	2.95 kg/m ³	0.7	Lognormal
C_r^d (chloride threshold concentration)	3.35 kg/m^3	0.375	Normal
Model error (diffusion coefficient)	1.0	0.2	Normal
f _{py} (steel yield strength)	1565 MPa	0.01	Lognormal
Model error (stress loss)	1.0	0.3	Normal
E_s (steel modulus elastic)	195000 MPa	0.02	Normal
Dead load (precast)	$1.03D_n^e$	0.08	Normal
Dead load (RC deck)	1.05D _n	0.1	Normal
Asphalt	90 mm	0.25	Normal
Single truck live load	275 kN	0.41	Normal
Model error GDF (Girder Distribution Factor)	0.93	0.12	Normal
Dynamic Load Factor	0.15	0.8	Normal
a \dot{F}_{c} = specified compressive concrete strength	^b Truncated at	t 10 mm	

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^d Truncated at 0.35 kg/m³

 c C_{nom} = nominal concrete cover $^{e}D_{n}$ = nominal dead load

Monte-Carlo event-based simulation analysis is used to simulate the spatially variable deterioration process and its effect on flexural capacity of the PC bridge girder. The analysis considers the variability and uncertainty of loads, material properties, dimensions and deterioration processes. For each simulation run the time to corrosion initiation and corrosion rate for each strand is calculated. At each annual time increment the peak annual dead plus live load is generated and strand stresses calculated from the FEM. If an actual strand stress is higher then the failure stress for any element (which will vary spatially along the length of the strand and decrease with time due to strand corrosion) then the strand is removed from that element and stresses in the remaining strands recalculated (i.e., stress redistribution). The event-based process continues for successive time increments, leading to more strand failures until the flexural capacity of the bridge girder is exceeded.

5 RESULTS

Figure 4 shows the timing of individual strand failures, for realisations from five Monte-Carlo simulation runs. It is observed that progressive failure of strands can take some time, but once several strands fail, then the resulting increase in strand stress due to load redistribution means than additional strand failures becomes more likely.

Figure 5 shows the distribution of time to failure of the AASHTO Type IV bridge girder, for 300 simulation runs. In some (rare) cases failure may occur within 50 years of service, however, the mean time to failure is 95 years. Figure 5 can readily be transformed into a plot of cumulative time-dependent probabilities of failure, see Figure 6. It should be mentioned here that the mean corrosion rates of the strands in the first and second level of reinforcement are approximately 1.75 μ A/cm² and 0.87 μ A/cm² as obtained from the corrosion rate model developed by Vu and Stewart (17).



Figure 4: Timing of individual strand failure for PC bridge girder in the middle of the span



Figure 5: Time of failure of PC bridge girder



Figure 6: Time-dependent probability of failure of PC bridge girder

5.1 The effect of corrosion rates

Figure 7 shows the effect of different corrosion rates on time dependent probability of failure of PC bridge girder, by comparing corrosion rates with those used in the development of Figure 5 and 6. As can be seen from the graph, the mean time of failure (Probability of Failure $p_f \approx 0.5$) increases from 95 years to 167 years if the corrosion rates reduces by 75 %. On the other hand, doubling the corrosion rate will reduce the mean time to failure from 95 years to 63 years. Not surprisingly, this show that corrosion rates is one of the primary variables that will affect the time to failure for a girder. If there is no deterioration then the probability of failure is negligible.



Figure 7: The effect of corrosion rates on time-dependent probability of failure of PC bridge girder

5.2 The effect of different failure criterion

Although experimental results from this study have shown that yielding with reduced ultimate failure strain is the appropriate mode of failure (Darmawan and Stewart (5)), there is however some uncertainty about the validity of the failure criterion. For example, literature surveys reveal that stress corrosion cracking (SCC) may occur in older structures which use an "old type" quenched and tempered prestressing steel (Mietz (11)). This type of steel was used mostly in Germany before 1960 but from 1965 the steel composition of this steel was modified to increase its resistance to SCC. Therefore, the probabilistic pitting model is combined with three different failure criterion (i.e. yielding f_{pv} , brittle fracture K_C and stress corrosion cracking K_{ISCC}) to investigate their effect to the time to failure for a girder. The statistical parameters used are given in Table 3. The results are presented in Fig. 8, which shows that the failure criterion influences significantly the time to failure for a girder. If SCC was the actual mode of failure then this analysis shows a mean time to failure of less than 25 years, which is less than the operating life of many existing PC bridges. During this time very small number of failures have been recorded, suggesting that a failure criterion based on SCC is overly conservative. However, more work is needed to ensure that the failure criterion used is appropriate for any given PC structural element.

Table 3:	Distribution	parameters	of	random	variables
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No	Variable	Distribution	Mean value	COV
1	K _{ISCC}	Lognormal	$43 \text{ MPa m}^{0.5} (9)$	0.10
2	K _C	Lognormal	86 MPa m ^{0.5} (9)	0.05



Figure 8: The effect of different failure criterion on time-dependent probability of failure of PC bridge girder

6 CONCLUSION

This paper described the development of probabilistic models to predict the spatial distribution of maximum depths of pitting for prestressing strands subjected to pitting corrosion. The models were then applied to a typical prestressed concrete bridge girder located at a coastal environment. The influence of pitting corrosion on structural strength,

mean time to failure and structural reliability were estimated. From this study, it was found that corrosion rates and the failure criterion have a significant effect to the time to failure for girder. The probabilistic approach developed in this study also allows for a more realistic representation of service life prediction than conventional deterministic approach.

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