Pitting Corrosion Model for Partial Prestressed Concrete (Pc) Structures in a Chloride Environment

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***Abstract* — Prestressed concrete structures cannot escape from corrosion problems, especially when they are subjected to very aggressive environment, such as chloride environment. For prestressed concrete structures, corrosion of prestressing strands may initiate structural collapse due to higher stress levels in the steel. Research on corrosion effect on concrete structures has mainly considered the effect of corrosion have on reinforced and full prestressed concrete structures. In this study, a structural framework will be developed to predict the flexural strength of partial prestressed concrete structures in a chloride environment. The framework developed will be combined with probability analysis to take into account the variability of parameters influencing the corrosion process.**

Keywords — **corrosion, partial prestressed concrete structure, chloride, probability.**

I. INTRODUCTION

Corrosion of reinforcing and prestressing steel due to chloride contamination can result in considerable reduction in service life of concrete structures. In general, corrosion is of most concern because of the associated reduction in steel cross-sectional area, cracking, spalling and loss of bond, which over time will lead to reductions of strength and serviceability of structures.

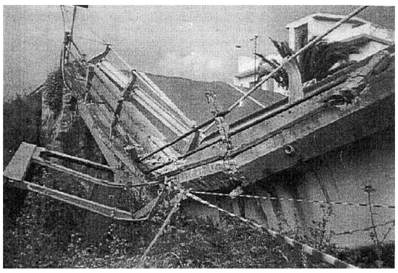

Fig. 1. The Saint Stefano Bridge in Sicily, Italy[1]

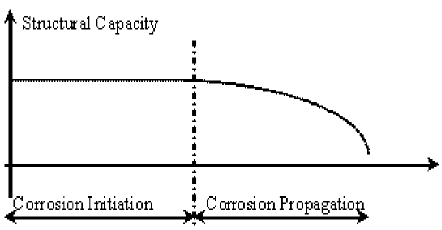

Fig. 2. Deterioration model

For prestressed concrete (PC) structures, the corrosion of prestressing strands may initiate structural collapse due to higher stress levels in the steel. Fig. 1 shows the collapse of The Saint Stefano Bridge in Sicily (Italy) which is located close to the sea. A post-mortem analysis of the collapsed bridge showed that the 40 year old post-tensioned bridge failed as a result of pitting corrosion near the box girder joint.

Chloride contamination is considered to be the major causes of corrosion of reinforced concrete (RC) and PC structures. This can occur either from the application of de-icing salts in cold regions or exposure to sea-spray in chloride environments. The deterioration of reinforced concrete structures due to chloride attack comprises of two stages. The first stage involves the movement of chlorides through concrete cover until they reach the threshold chloride concentration at the steel to initiate active corrosion (see Fig. 2). The second stage is called corrosion propagation, where reinforcing steel corrodes causing loss of steel area (metal loss) and therefore reduces structural capacity.

Different approaches have been made to model chloride penetration in concrete (i.e. corrosion initiation). The different approaches proposed clearly underline that the actual chloride penetration process is very complicated, and may involve a combination of processes contributing towards the overall ingress of chlorides [2].

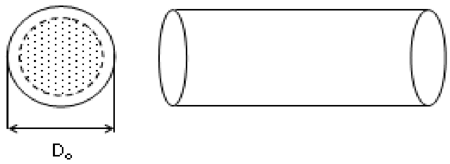

Fig. 3. General corrosion

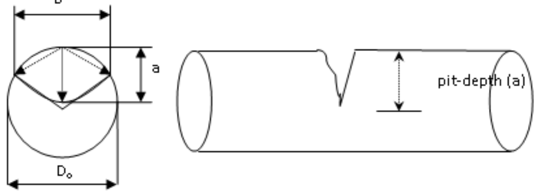

Fig. 4. Pitting corrosion and pit configuration [4]

Of all the available models, it is generally accepted that the model based on diffusion theory represents the chloride ingress in concrete. Hence, in this paper only corrosion propagation will be discussed.

Corrosion propagation is mostly modeled by assuming a relatively uniform loss of material thickness (see Fig. 3), such as used by Vu and Stewart [3]. However, this

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approach is not accurate for concrete structures subjected to chloride attack, which usually experiences pitting corrosion (see Fig. 4).

Darmawan and Stewart [5] has developed pitting corrosion model for PC structures subjected to chloride attack. This model was developed from accelerated corrosion test using four slabs, each of dimensions 1500 mm×1000 mm×250 mm with wires/strands. Using a similar approach, later on Darmawan [6] has also developed pitting corrosion model for RC structures subjected to chloride attack. From these tests, it was found that the distribution of maximum pit-depths for prestressing wires and reinforcing bar is best represented by the Gumbel (EV-Type I), see Fig. 5 and 6 for Inverse Cumulative Distribution Function (CDF-1) plots. Note that in these Figure, icorr-exp is the corrosion rate used in the test, To-exp is the length of the test, Lo is the length of wire/rebar where pit-depth is measured, and n represent the number of pit-depth measured after the corrosion test (number of pit-depth sample).

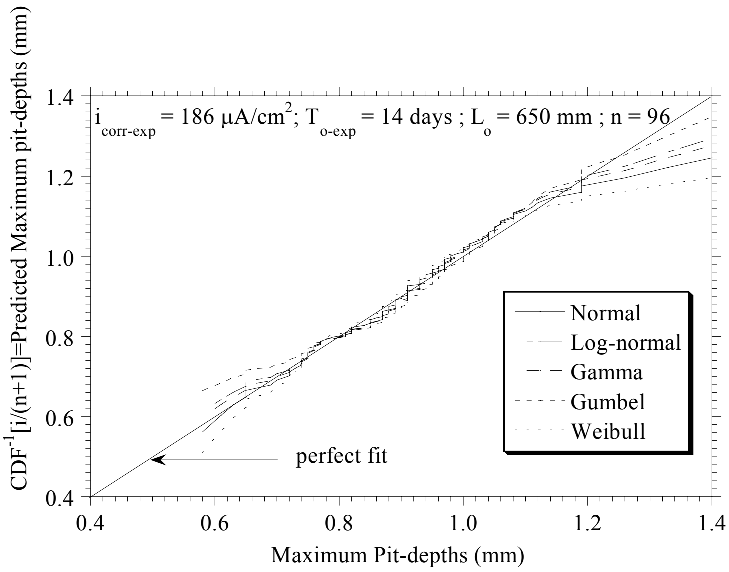

Fig. 5. Inverse CDF (CDF^{-1}) plots for maximum pit-depths in prestressing wires

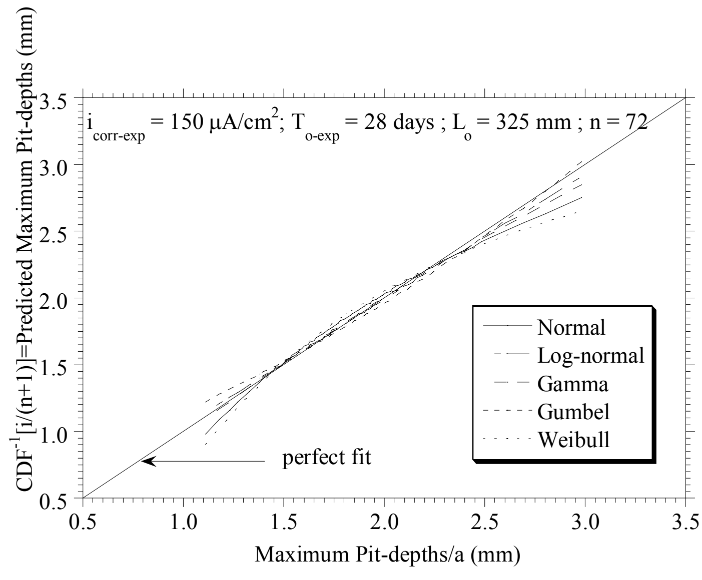

Fig. 6. Inverse CDF (CDF^{-1}) plots for maximum pit-depths in a reinforcing bar

II. Corrosion Model For Partial Pc Structures

Research on corrosion effect on concrete structures has so far considered only the effect of corrosion have on reinforced and full prestressed concrete structures. In this study, a structural framework will be developed to predict the flexural strength of partial prestressed concrete structures in a chloride environment. Note that in partial prestressed concrete structures, both non prestressing steel (passive) and prestressing (active) reinforcement are utilized to carry the load. The corrosion model previously developed for reinforced and prestressed concrete structures will be combined to determine the effect of corrosion has on partial prestressed concrete structures. The framework developed will be combined with probability analysis to take into account the variability of parameters influencing the corrosion process. This approach allows more accurate prediction of service life of partial prestressed concrete structures in a chloride environment.

The following assumptions are made in developing a more general probabilistic model for pitting corrosion: homogeneous environment along the wire/rebar under consideration (corrosion rate assumed constant along wire); after an initial period of corrosion, the number of pits formed is assumed constant, length of pit is held constant and pit depth continues to increase; and at any cross-section of the wire only one pit can form.

The predicted Gumbel distribution of maximum pit depth (a in mm) at any time T (years), corrosion rate $i_{corr}(1)$ in $\mu A/cm^2$ at start of corrosion propagation and wire/rebar length L (mm) is thus:

$$f_a(T, i_{corr}, L) = \frac{\alpha}{\lambda^{0.54}} e^{-\alpha\left(\frac{a}{\lambda^{0.54}}-\mu\right)} e^{-e^{\alpha\left(\frac{a}{\lambda^{0.54}}-\mu\right)}} \qquad T > T_i$$

(1) where,

$$\lambda = \frac{\left[D_o^2 - \left(D_o - 0.0232 i_{corr}(1)\left\{1 + \frac{\kappa}{\theta+1}\left[(T-T_i)^{\theta+1} - 1\right]\right\}\right)^2\right]}{\left[D_o^2 - \left(D_o - 0.0232 i_{corr}(1)\left\{1 + \frac{\kappa}{\theta+1}\left[T_o^{\theta+1} - 1\right]\right\}\right)^2\right]} \tag{2}$$

$$T_o = \exp\left[\frac{1}{(\theta+1)}\ln\left(\frac{(\theta+1)(i_{corr-\exp}T_{o-\exp}) + (\kappa-\theta-1)(i_{corr}(1))}{\kappa i_{corr}(1)}\right)\right] \tag{3}$$

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

$$i_{corr}(T-T_i) = i_{corr}(1)\times\kappa(T-T_i)^{\theta} \qquad T-T_i \geq 1\,year \tag{5}$$

T_i is time to corrosion initiation (years). μ_{o-exp} and α_{o-exp} are the parameters of the Gumbel distribution as obtained from statistical analysis of maximum pit depths recorded from the accelerated corrosion tests (see Table 1 and 2), L_{o-exp} is the length of wire/rebar used in the accelerated corrosion tests, D_o is the initial diameter of the wire/rebar (mm), and κ and θ are corrosion rate empirical factors. If corrosion rate reduces with time then κ=0.85 and θ=-0.29[3]. Otherwise, if corrosion rate is constant with time (time-invariant) then κ=1 and θ=0. The geometric model proposed by Val and Melchers[4] is then used to predict the loss of cross-sectional area for a pit size of depth a, see Fig. 4. Full development of equations 1 to 5, see Darmawan & Stewart [5].

III Statistical Parameters of Maximum Pit-Depths Distribution

The maximum pit-depth of corroded prestressing or reinforcing steel is an important parameter as it is the likely place of critical (minimum) section of the steel. Therefore, it is also the likely place, where failure of the steel occurs. Statistical parameter of maximum pit-depth distribution used in this study is given in Table 1 and 2 for prestressing strands [5] and reinforcing bars [6], respectively. These s tatistical parameters were obtained from accelerated corrosion tests using concrete slabs, each of dimensions 1500 mm×1000 mm×250 mm with strands/rebar. The accelerated corrosion process was introduced to the rebar using an electric current, which was induced from a power supply through a current regulator. At the completion of each corrosion test the specimen was broken up and the steel then cleaned, dried and weighed using the method as specified by Standard Practice for Preparing, Cleaning, and Evaluating Corrosion Test Specimens [7]. The pit-depth in the corroded steel was then measured using a micrometer gauge.

The pitting corrosion statistical parameters μ_{o-exp} and α_{o-exp} are indicative only and increased confidence in predictions will be obtained if these parameters are based on tests which more closely represent field conditions-that is, longer T_{o-exp} and lower $i_{corr-exp}$.

IV. Statistical Parameters Of Partial PC Structure

Using probabilistic analysis, the statistical parameters of maximum pit-depths distribution are combined with statistical parameters of prestressed concrete beams (i.e. beam dimension, concrete strength, steel yield strength, cover thickness, insitu strength factor, model error for flexure and corrosion rate) to determine the effect of corrosion on flexural strength of partial PC beam. The statistical parameters of PC beams used in the probabilistic analysis are given in Table 3. Table 3 shows that the parameters influencing flexural strength of partial prestressed concrete beam have some uncertainty (random variables). These parameters have coefficient of variations of 0.025 to 0.2. For pitting model parameter, the coefficient of variation is also on the high side (e.g. 0.17 and 0.23), see Tables 1 and 2. Therefore, flexural strength determination of partial PC beams based on deterministic analysis is inaccurate. Monte Carlo simulation will be used to determine the distribution of flexural strength of partial PC beams.

V. Flexural Strength Of Partial PC Structure

The flexural strength of partial prestressed concrete beam can be derived from the principal of equilibrium of section and strain compatibility between steel and concrete, as treated in most prestressed concrete design guides[15], see Fig. 7. It is assumed that ultimate load is reached when the concrete compressive strain in the extreme compressive zone equals 0.003 [16].

From Fig. 7, the ultimate flexural strength M_n at time T (in years) can be determined as

$$M_n(T) = F_1(T)d_1 + F_2(T)d_2 + F_3(T)d_3 - C_c d_c \tag{6}$$

$$F_1(T) = A_{ps}(T)f_p \tag{7}$$

TABLE 1
STATISTICAL PARAMETERS FOR THE MAXIMUM PIT-DEPTHS (A) OF PRESTRESSING STRAND

T_{o-exp} (years)	$i_{corr-exp}$ ($\mu A/cm^2$)	Length L_o (mm)	μ_{o-exp}	α_{o-exp}	a (mm) mean	a (mm) COV
0.0383	186	650	0.84	8.1	0.91	0.17

μ_{o-exp} and α_{o-exp} are the Gumbel parameters

TABLE 2
STATISTICAL PARAMETERS FOR THE MAXIMUM PIT-DEPTHS (A) OF REINFORCING BAR

T_{o-exp} (years)	$i_{corr-exp}$ ($\mu A/cm^2$)	Length L_o (mm)	μ_{o-exp}	α_{o-exp}	a (mm) mean	a (mm) COV
0.0767	150	325	1.68	2.99	1.87	0.23

TABLE 3
STATISTICAL PARAMETERS FOR PC BRIDGE GIRDER [8]

Parameters	Mean	COV	Distribution	Reference
f'_{cyl} concrete cylinder strength	$F'_c{}^a$ + 7.5 MPa	$^b\sigma$=6 MPa	Lognormal	Stewart[9]
k_i in-situ concrete strength factor	1.2-0.0082×mean(f'_{cyl})	0.1	Normal	Stewart[9]
f_{py} yield strength	0.88 $f_{pk}{}^c$	0.025	Normal	Mirza, et al.[10]
f_y	465 MPa	0.1	Beta	Mirza and McGregor[11]
C_b bottom cover	C_{bnom}	σ = 7.9 mm	Normal	Mirza and MacGregor[12]
H beam depth (mm)	H_{nom}+0.8	σ= 3.6 mm	Normal	Mirza and MacGregor[12]
B (Beam Width)	B_{nom}+2.5 mm	σ= 3.7	Normal	Mirza and McGregor[12]
^{d}ME (Flexure)	1.01	0.046	Normal	Ellingwood et al. [13]
ME - corrosion rate (i_{corr})	1	0.2	Normal	Vu and Stewart[14])

^{a} F'_c=specified (characteristic) concrete compressive strength; $^b\sigma$= standard deviation; $^c f_{pk}$ = characteristic tensile strength of prestressing steel; ^{d}ME= Model Error

$$F_2(T) = A_{ps}(T)f_p \quad (8)$$
$$F_3(T) = A_s(T)f_s \quad (9)$$

Where, $F_1(T)$, $F_2(T)$ and $F_3(T)$ are the forces in the prestressing strands or reinforcing steels at time T, C_c is the force in the concrete compressive zone and d_1, d_2, d_3 and d_c are the distances from the application of each force to the top of the concrete section. $A_{ps}(T)$ and $A_s(T)$ are the prestressing steel and reinforcing steel area at time T, and f_p and f_s are the stress in the prestressing steel and reinforcing steel, respectively. As the time since corrosion (T) increases, the corrosion will decrease the steel area and therefore reduce the flexural strength of partial PC beam.

VI. RESULTS AND DISCUSSION

For illustrative purpose, the corrosion model developed will be used to determine the effect of corrosion has on partial PC beam shown in Fig. 8[15]. The prestressing steels (A_{ps}) comprise two cables, each consisting of 18 super grade 7-wire strands of 12.5 mm diameter, whereas passive reinforcement (A_s) comprise of 10 reinforcing bar of 32 mm diameter. The beam is exposed in a near-coastal area. Four different scenarios of corrosion are considered: no corrosion, only reinforcing bars corrode, only prestressing steels corrode, both reinforcing bar and prestressing steel corrode

The corrosion rate used in the analysis is 1 $\mu A/cm^2$ (0.012 mm/year), which can be classified as low to moderate corrosion rate [17].

For no corrosion case, the histogram of flexural strength of PC beam (Mn) obtained from Monte Carlo simulation is shown in Fig. 9. Fig. 9 show that for no corrosion case the flexural strength has a mean value of 666.8 ton-m, with coefficient of variation of 4.44%. For the second case (only rebars corrode), the histogram of flexural strength of PC beam (Mn) after 40 years since corrosion initiation is shown in Fig. 10. Fig. 10 shows that compared with no corrosion case, the mean flexural strength has decreased from 666.8 ton-m to 560.6 ton-m (15% reduction).

For the third case (only prestressing steel corrode), the histogram of flexural moment capacity of PC beam (Mn) after 40 years since corrosion initiation is shown in Fig. 11. Fig. 11 shows that compared with no corrosion case, the mean flexural strength has decreased from 666.8 ton-m to 494.5 ton-m (26% reduction).

For both passive and active reinforcement corrode, the histogram of flexural strength of PC beam (Mn) after 40 years since corrosion initiation is shown in Fig. 12. Fig. 12 shows that compared with no corrosion case, the mean flexural strength has decreased from 666.8 ton-m to 486.5 ton-m (28% reduction).

Fig. 13 demonstrates the effect of corrosion has on flexural strength of PC beam, by assuming that both prestressing steel and reinforcing bar is corroding. The Fig. shows that with time the flexural strength of the beam decreases considerably. For example after 40 years since corrosion initiation, the probability of flexural strength less than 500 ton-m is around 50%. For no corrosion case, the probability of flexural strength less than 500 ton-m is zero. This Fig. clearly indicates that corrosion has significant effect on flexural strength of PC beam.

For high strength steel, such as prestressing steel, there is high possibility to have different mode of failure than yielding. It is known from literature [18] that with increasing strength of the steel, the possibility to have fracture or stress corrosion cracking (SCC) as mode of failure of the steel also increases. Furthermore, the presence of pitting with various depths and shapes may further raise the possibility of stress concentration associated with pit geometry, leading to brittle failure (not yielding). Hence, Fig. 14 shows the effect of different modes of failure on the flexural strength of the beam after 10 years since corrosion initiation. The Fig. indicates that different mode of failure of prestressing steel effect significantly the flexural strength of corroded partial prestressed concrete beam. For example, after 10 years since corrosion initiation time, assuming Stress Corrosion Cracking (SCC) as mode of failure of prestressing steel reduce the mean flexural strength by 60%, while assuming yielding as mode of failure only reduce the mean flexural strength by 10%. Note that as the corrosion progress, the probability of having SCC as mode of failure will also increase.

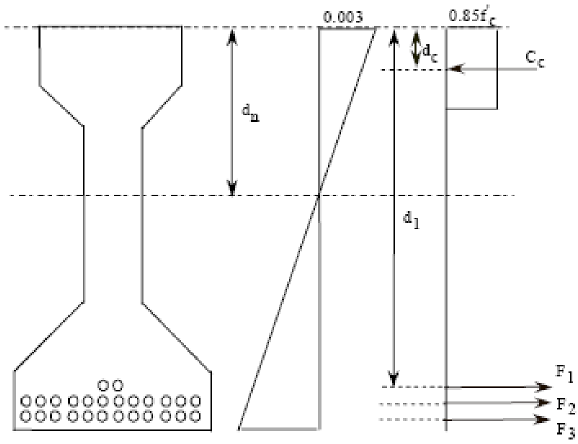

Fig. 7. Forces and strain diagram at ultimate flexural strength

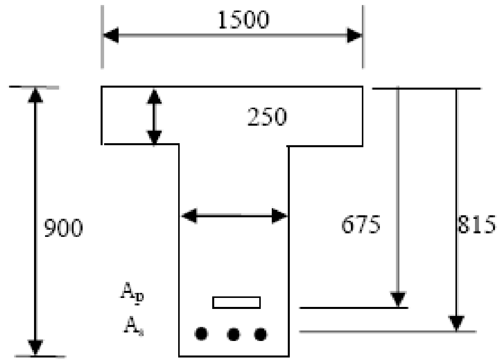

Fig. 8. Prestressed concrete T-beam

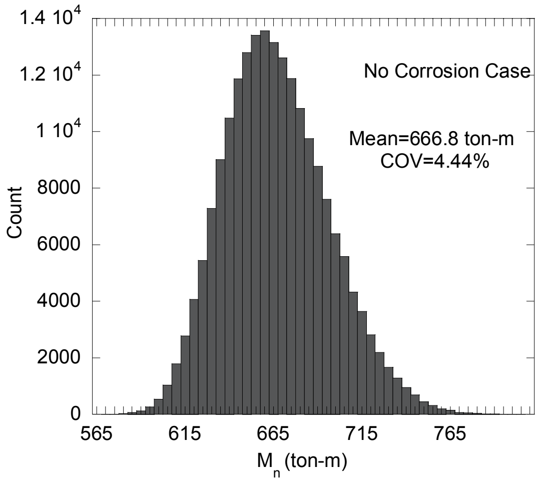

Fig. 9. Flexural strength for no corrosion case

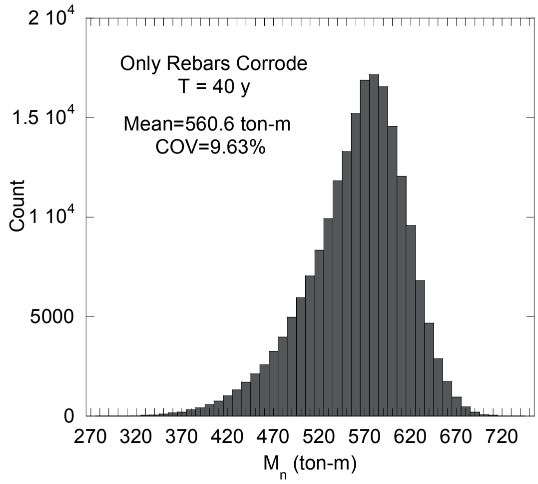

Fig. 10. Flexural strength for corrosion at reinforcing bars

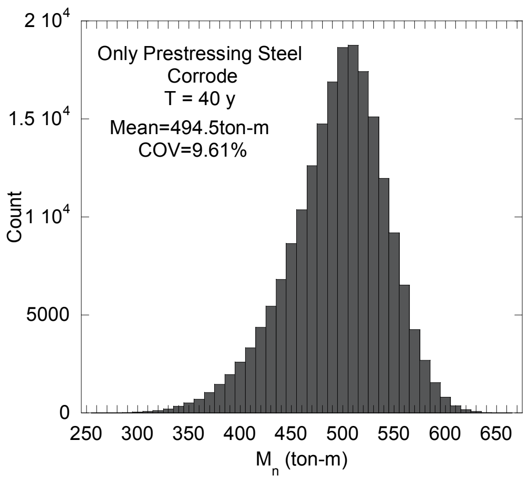

Fig. 11. Flexural strength for corrosion at prestressing steel

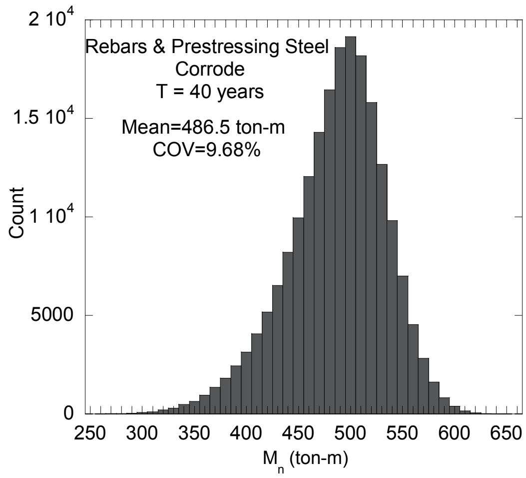

Fig. 12. Flexural strength for corrosion at reinforcing and prestressing steel

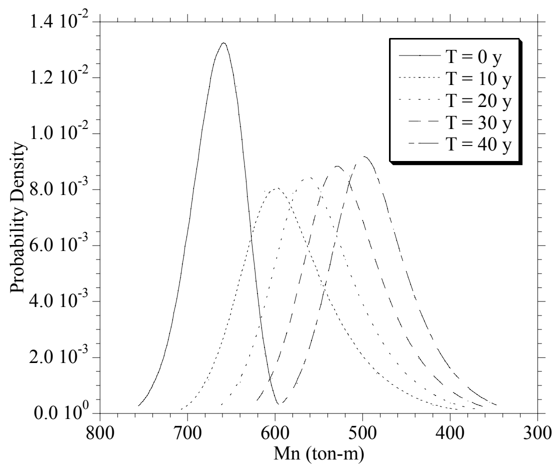

Fig. 13. Distribution of flexural strength with time

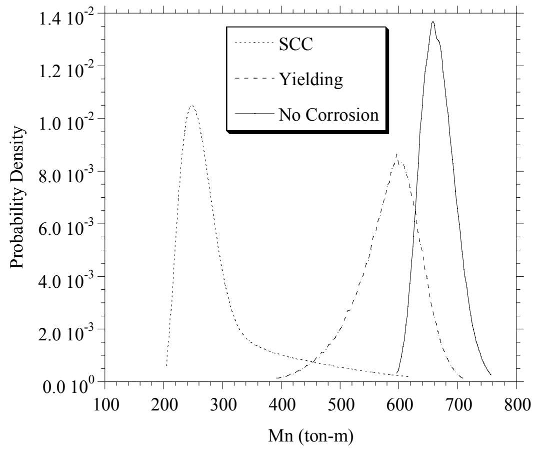

Fig. 14. Effect of different mode of failure of prestressing steel on flexural strength

VII. CONCLUSION

This paper has described the structural framework to predict the service life of partial PC structure in a chloride environment. The framework combined the pitting corrosion model previously developed for reinforcing bar and prestressing wires to determine the effect of corrosion on flexural strength of PC beam. The framework also considers the variability of concrete properties of partial PC beam in the analysis using probability analysis. This will allows accurate prediction of service life of partial PC structure in a chloride environment. From the analysis it can be concluded that corrosion has significant effect on flexural strength of partial PC beam.

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