Sixth International Conference on Durability of Concrete Structures **Paper Number DSM05**
18 - 20July 2018
University of Leeds, Leeds, West Yorkshire, LS2 9JT, United Kingdom

Residual Service Life Estimation and its Importance for Pretensioned Concrete (PTC) Bridges in Coastal Cities

D. Joseline, D. Kamde, S. Rengaraju and R. G. Pillai
Indian Institute of Technology Madras, Chennai, India

ABSTRACT

Many pre-tensioned concrete (PTC) bridges are experiencing premature chloride-induced corrosion. Hence, it is crucial to estimate their residual service life and update it with newer data on a periodic basis - to plan for corrosion prevention or control measures and ensure safety of existing bridges. Critical chloride threshold (Cl_{th}) is one of the parameters necessary to estimate the corrosion initiation period. However, quantitative estimates on Cl_{th} for prestressing (PS) steel are not well-reported in literature. This paper presents experimental data on the Cl_{th} of PS steel, the chloride diffusion coefficient (D_{cl}), and surface chloride concentration (C_s) of concrete samples obtained from a PTC bridge girder in a coastal city in India. For estimation of Cl_{th}, 5 specimens were cast with PS steel wires embedded in ordinary Portland cement (OPC) mortar containing 30% of Class F fly ash (similar composition as that of the bridge girder). They were cured for 28 days and then subjected to cyclic wet-dry exposure using simulated concrete pore solution containing 3.5% sodium chloride. (Linear polarization resistance (LPR) tests were performed at the end of each exposure cycle, corrosion initiation was detected using statistical methods, and Cl_{th} was determined). Using the determined Cl_{th} and D_{cl}, and other relevant parameters, the cumulative distribution functions of time to corrosion initiation was developed. It was found that the average time to corrosion initiation was about 40 years, whereas the structure was designed for 120 years. Also, it was estimated that the corrosion products will accumulate within the interstitial space between the 7 wires in a strand and will not flow through the concrete cover and reach the concrete surface (showing rust stains) until about 5% of strand is corroded (about 10 years). This indicates a dire need for regular data collection, updating the residual life estimates, which will help in developing corrosion prevention strategies for PTC structures.

Keywords: Residual service life; prestressed concrete; corrosion; chloride threshold, diffusion coefficient, preventive maintenance

1.0 INTRODUCTION

Many of the existing long span bridges consist of pre-tensioned concrete (PTC) elements. Typically, these structures are expected to be serviceable for 100+ years. However, premature corrosion of 7-wire strands has been observed in some young PTC bridges, even necessitating the replacement of the entire superstructure in one of them at just about 40 years of exposure to chloride environments. As the strands are constantly under tensile stress, the probability of occurrence of localized/pitting corrosion is very high. Fig. 1 shows the severely corroded wires (exhibiting localized corrosion) extracted from a strand in a 40-year old bridge girder in New Zealand (NZTA Report 502, 2003). This can lead to serious consequences – indicating the need for residual life estimation and corrosion prevention measures.

Service life of reinforced concrete structures comprises of corrosion initiation and corrosion propagation phases. Corrosion initiation period (T_i) is the time taken for sufficient chloride to build up at the surface of steel and initiate corrosion. This can be estimated using Fick's 2nd law of diffusion, if the chloride concentration at the surface (Cl_s), rate of diffusion of chlorides through the concrete (D_{Cl}), and the amount of chloride necessary to initiate corrosion (Cl_{th}) are known.

Fig. 1. Severe localized corrosion on prestressed wires extracted from about 40-year old Bridge in New Zealand (NZTA report 502, 2003)

It is well-reported that in PTC structures, corrosion induced failures are catastrophic in nature. In other words, once corrosion gets initiated, it progresses rapidly with no visible indication and can result in sudden failure (Lau and Lasa, 2016). Thus, for PTC structures, the corrosion initiation period is a safer measure of service life. Throughout this paper, 'service life' denotes the time taken for the initiation of strand corrosion (T_i).

Chloride diffusion coefficient (D_{Cl}) is a parameter that can vary with the type/ permeability of concrete used. Nowadays, concrete of various grades is used to construct PTC bridges. Generally, a less porous concrete is expected to have a higher strength as well as durability. ACI 318 (1995) specifies a minimum compressive strength limit for different exposure conditions. However, it is also reported that concrete mixes with similar compressive strengths need not have similar permeability properties (Armaghani *et al.*, 1992). It is therefore advisable to obtain this parameter from field investigations by testing core samples obtained from the structure to be assessed. Also, the data collected from the field specimens will reflect the realistic behavior of concrete; and hence, can lead to better residual life estimates.

Corrosion initiation in any steel-binder system is governed by its Cl_{th}. Literature provides sufficient information on the Cl_{th} of conventional reinforcing bars. However, very limited information is available on the Cl_{th} of prestressing steel strand. Table 1 provides a summary of these reported values and indicate significant variation (as reported in FHWA-HRT-12-067 (2012).

Table 1. Summary of chloride threshold for prestressing (PS) steel embedded in OPC systems (FHWA-HRT-12-067 (2012))

Chloride threshold of PS steel (%by weight of binder)	Reference
0.11 to 0.17	Stark (1984)
1.8 to 2.2	Lukas (1985)
1.2 to 1.4	Pfeifer *et al.* (1987)
0.08	Wang et al (2005)
0.04 to 1.2	Azuma *et al.* (2007)
0.006 to 0.018	Trejo *et al.* (2009)

These values did not consider the effect of stress and supplementary cementitious materials (SCMs). Also, to shorten the test duration, some of these tests employed the use of external voltage application to accelerate the chloride ingress, which is not representative of the actual mechanisms. Also, the effects of chemical composition on the corrosion resistance of prestressing steel is not well reported.

Polder et al (2012) collected field data and estimated the huge, anticipated repair work of conventionally reinforced concrete (CRC) bridges. Similar data on PTC bridges are not available. However, many premature failure instances of PTC bridges have been reported (see Table 2). Unfortunately, many bridge agencies are still adopting the corrective maintenance strategies. There is a need to highlight the need for abolishing the corrective maintenance approach and implementing preventive maintenance approach – to extend the residual service life with minimal economic burden.

Table 2. Summary of instances of premature corrosion in pre-tensioned bridges

Details of the bridge (*Reference*)	Visible Signs	Observation upon detailed investigation	Result of delayed detection and (the age at the time of detection)
Tiwai Point Bridge, New Zealand (*NZTA Report 502, 2003*)	Cracking	Upto 60% estimated section loss	Decommissioning (40 years)
Boundary Creek Bridge, New Zealand (*NZTA Report 502, 2003*)	Cracking	Localized wire breakage	Strand removal (48 years)
Hamanatua Stream Bridge, New Zealand (*Bruce et al.,2008*)	Spalling of cover concrete	Upto 10% estimated section loss	Significant repair (38 years)
A bridge in Gulf of Mexico (*Novokshchenov,1989*)	Corrosion at expansion joints	Leakage of chloride-laden moisture from deck	*Not reported* (16 years)
A box-girder viaduct (*Novokshchenov, 1989*)	Corrosion at expansion joints	Leakage of chloride-laden moisture from deck	*Not reported* (29 years)
Lowe's Motor Speedway Pedestrian Bridge, North Carolina (*Sly, 2001*)	Sudden collapse	pre-mixed Calcium Chloride set accelerator	Human injury (5 years)

2.0 RESEARCH SIGNIFICANCE

Many pre-tensioned concrete (PTC) bridges are relatively young when compared to conventionally reinforced concrete (CRC) bridges. The number of cases of corrosion observed on the young PTC bridges are less than that observed in older CRC bridges. Hence, there is a general belief that PTC bridges are more durable. However, more recently many PTC bridges have started showing signs of premature corrosion (i.e., much before their target or design service life). One approach to avoid such premature corrosion is to conduct frequent field inspection and testing and implement preventive maintenance measures. Also, unlike CRC bridges, PTC bridges may not show any sign of corrosion (like rust stains) until significant corrosion has occurred. This paper focusses on these aspects and suggests a frequency of data collection for updating the residual service life estimate and implement preventive maintenance measures on PTC bridges.

3.0 EXPERIMENTAL WORK

The objective of this study was to estimate the residual service life (RSL) and develop an inspection strategy for a PTC bridge in a coastal city in India. A six-year old PTC bridge girder was chosen. The D_{Cl} and Cl_s were determined from core samples obtained from the girder. The Cl_{th} was estimated for a similar prestressing steel-binder system from a laboratory study based on linear polarization resistance technique. Then, the cumulative distribution function of service life of the PTC girder was determined based on the determined D_{Cl}, Cl_s, Cl_{th}, and the Fick's 2^{nd} law of diffusion. Also, the mechanism of strand corrosion, difficulties in detecting corrosion initiation using visual observations, and the importance of assessment and updating of residual life estimates, especially for PTC systems are discussed.

3.1 Estimation of chloride diffusion coefficient

A 6-year-old bridge situated at a coastal city in India was chosen for the study. The bridge consists of PTC girders with M60 grade concrete (mix ratio 1:1.13:2.10) of composition as described in Table 2.

Table 3. Details of concrete used in the bridge

Constituent	Content
Cement (Kg/m^3)	440
Flyash (Kg/m^3)	125
20 mm aggregate (Kg/m^3)	500
12.5 mm aggregate (Kg/m^3)	687
Manufactured sand (Kg/m^3)	640
Water (Kg/m^3)	183
Superplasticiser (%)	1.1
Retarder (%)	0.1

Diffusion coefficient

At about 6 years after construction, three cylindrical concrete samples of approximately 90 mm diameter and 80 mm length were extracted from the PTC bridge girder located in a coastal city in India. Following the ASTM C1556 (2016) and SHRP S 330 (1992) procedures, chloride profiles of each cylindrical specimen were obtained. Using Fick's second law of diffusion (Eq. 1), the chloride diffusion coefficient was determined.

$$\mathrm{C(x,t)} = \mathrm{C_s} - (\mathrm{C_s} - \mathrm{C_i}) \times \mathrm{erf}\left(\mathrm{x}/\sqrt{4 \times \mathrm{D}_{cl} \times \mathrm{t}}\right) \qquad (1)$$

where, (x, t) is the chloride concentration measured at depth, d, from the exposed concrete surface at exposure period, t; C_s is the surface chloride concentration; 'C_i' is the initial chloride concentration, D_{cl} is the chloride diffusion coefficient; and erf() is error function. D_{Cl} was determined as follows.

$$\mathrm{D(t)} = \mathrm{D_{28}}(t_0/\mathrm{t})^{\mathrm{m}} \qquad (2)$$

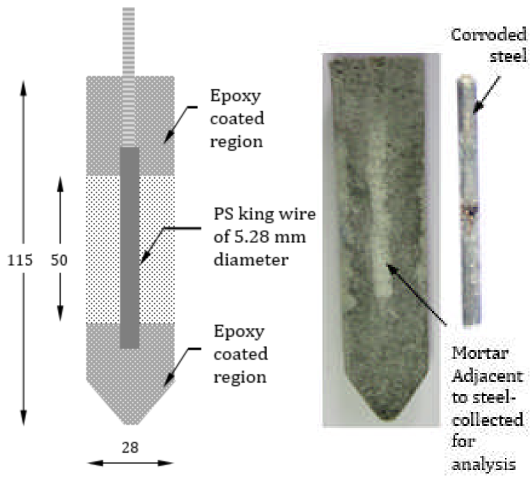
where, D(t) is the chloride diffusion coefficient (D_{Cl}) at time t (here, 6 years), D_{28} is chloride diffusion coefficient of concrete at the age of 28 days, t_o is equal to 28 days, t is the age of concrete when D_{Cl} was estimated, and m is decay constant. Decay constant (m) was taken as 0.6 for fly ash-based concrete (Gettu *et al.* 2018).

Maximum Surface Chloride Concentration

It is assumed that each concrete can accumulate a specific maximum concentration of chloride on the surface, termed as C_{max}. In this study, Rapid Migration test (NORD NT-BUILD 492 (1999)) was performed and the amount of chlorides accumulated at the surface (top 5 mm layer) of the specimens at the end of the test was considered to be the maximum chloride content (C_{max}) that can possibly be available under normal exposure during service life.

3.2 Estimation of critical chloride threshold

Lollipop type specimens were used for performing chloride threshold study (see Fig. 2).



Dimensions are in 'mm'

Fig. 2. Details of the lollipop type corrosion test specimen used in the study

Central king wire (5.28 mm diameter) extracted from 7-wire prestressing strands of nominal diameter 15.2 mm was embedded in a mortar cylinder. The mortar was made with binder (OPC with 30% Class F Fly ash). The composition was kept similar to that of the concrete used in the PTC girder being assessed. A water:binder:sand ratio of 0.5:1: 2.75 was adopted for preparing the mortar.The chemical composition of the PS steel and cements used in the study are presented in Table 4. It should be noted that the Cu, Al, Ni, and Cr contents are smaller and the phosphorous content is larger than that in the prestressing steel available in the USA (Pillai, 2009). The specimens were cured for 28 days and then subjected to cyclic dry-wet exposure (5-day drying followed by 2-day wetting) using simulated concrete pore solution (with 0.3 g $Ca(OH)_2$ + 10.4 g NaOH + 23.2 KOH and 967 g H_2O per litre of solution) containing 3.5% NaCl.

For the electrochemical testing, a 3-electrode system was used. In this, the embedded PS steel wire was the working electrode, a cylindrical Nichrome mesh encapsulating the lollipop specimen was the counter electrode and saturated calomel electrode was the reference electrode. The simulated concrete pore solution containing 3.5% NaCl was used as the electrolyte. Linear Polarisation Resistance (LPR) tests were performed after every wet period, using Solartron 1287potentiostat over a scan range of ±15mV with respect to the open circuit potential, at a scan rate of 0.1667 mV/s.

Table 4. Chemical composition of materials used for Cl_{th} determination

Steel		Cement		
Element	Weight (%)	Oxides	OPC	Class F Fly ash
Cu	0.02	CaO	64.59	1.28
Co	0.01	SiO_2	19.01	59.32
Al	0.04	Al_2O_3	4.17	29.95
Ni	0.02	Fe_2O_3	3.89	4.32
Cr	0.27	MgO	0.88	0.61
P	0.06	Na_2O	0.16	0.16
Mn	0.83	K_2O	0.59	1.44
Si	0.29	TiO_2	0.23	-
C	0.84	SO_3	1.70	0.16
Fe	Remaining	LOI	1.40	-

The LPR curve, which is a plot of instantaneous overvoltage against instantaneous current density, was generated at the end of every wet period and the polarisation resistance R_p was determined. A statistical procedure was followed to detect the corrosion initiation. For the statistical analysis, the plot of $1/R_p$ Vs. duration of exposure was considered. When five consecutive values of $1/R_p$ lie within a boundary of M±1.3SD, the system was considered to have stabilised (M - mean; SD - standard deviation). Following this stable state, if two future readings lie above (M+3SD), corrosion is said to have initiated. Atypical plot showing the variation in R_p is shown in Fig. 3.

Upon corrosion initiation, the specimens were split at the level of steel and the mortar adjacent to the steel was powdered and collected. The autopsied test specimen and the sample collection location are shown in Fig.2. Chloride analysis was done as per the procedure outlined in SHRP S-330 (1992). The chloride content (% bwob - percent by weight of binder) was calculated and defined as the Cl_{th} of PS steel in OPC mortar with 30% Class F Fly ash.

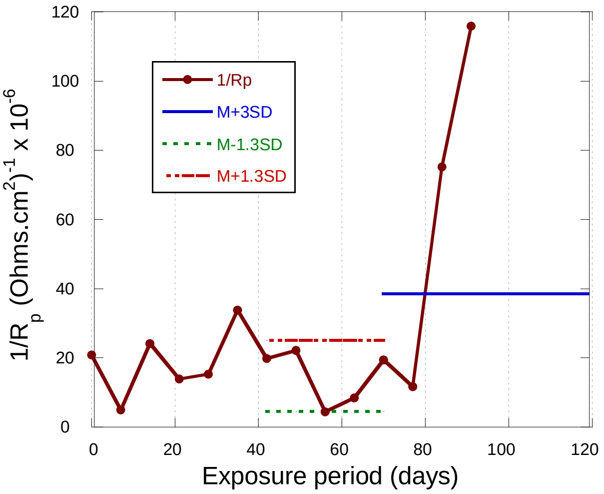
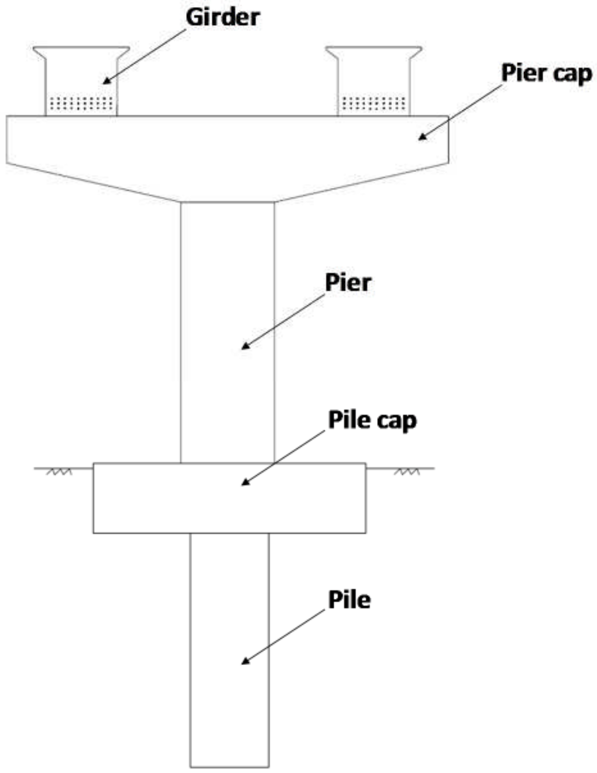


Fig. 3 Identification of corrosion initiation based on LPR test results

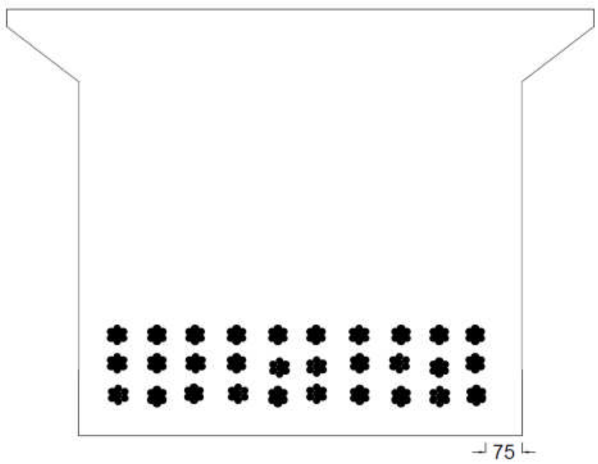
4.0 RESULTS AND DISCUSSION

The diffusion coefficient (D_{Cl}) and surface chloride (Cl_s) concentrations of the 3 core specimens collected from the 6-year old bridge were determined as described earlier. The mean D_{Cl} was found to be 4.98×10^{-11} m^2/s (51% COV). The maximum surface chloride concentration was found to be 5.11% by weight of binder (5% COV). The critical chloride threshold (Cl_{th}) was found to be 0.543% by weight of binder (16% COV). Assuming normal distribution, the obtained quantities were used as input for service life estimation. A schematic of the PTC bridge girder under study is shown in Fig. 4.The cover depth provided is 75 mm. A 10% COV was assumed for cover depth. Using this and the experimental results, a probabilistic estimate of the service life (*defined as time to corrosion initiation*) was obtained based on Fick's 2nd law of diffusion.

The cumulative distribution function (CDF) for the time to corrosion initiation is shown in Fig. . To illustrate the effect of D_{Cl}, CDFs were generated for cases with D_{Cl}, D_{Cl}+20%, and D_{Cl}–20%.The time for corrosion initiation corresponding to 50% probability are marked in Fig. . It can be inferred that the average residual service life of the bridge girder is only 36 years (42 minus 6). For cases with lower and higher D_{Cl}, it is seen that initiation has a 50% chance of occurrence in 23 and 110 years, respectively. This indicates that preventive maintenance must be planned soon (say, every decade), without which, chlorides might continue to ingress, corrosion might initiate and propagate insidiously leading to localized metal loss (as shown in Fig. 1). It is recommended that for this structure, chloride profiles must be monitored and preventive measures such as surface coating could be adopted every 10 years. Also, cylindrical specimens could be extracted, D_{Cl} be determined, and the current estimate of residual service life be updated with newer information.



(a) Elevation of the superstructure and substructure of the PTC girder



(b) Location of prestressing strands in the PTC girder

Fig. 4. Sectional details of the PTC bridge girder considered

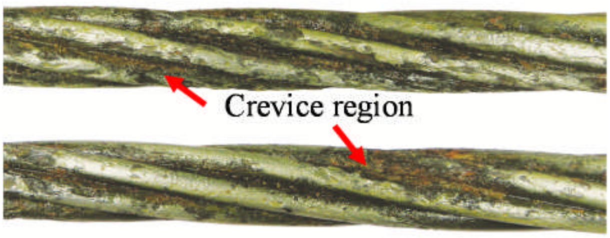


Fig. 5. PS strands extracted from G109 specimens after cleaning with G1 solution

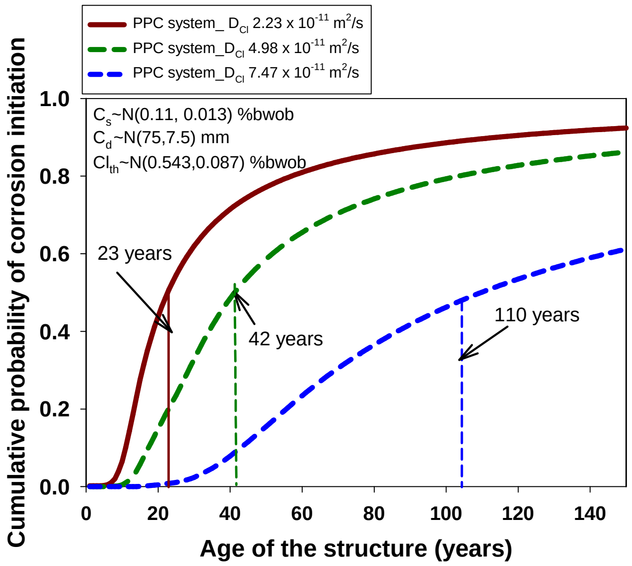


Fig. 6. Cumulative probability of corrosion initiation

5.0 IMPORTANCE OF UPDATING RESIDUAL SERVICE LIFE ESTIMATES

In PS strands, the corrosion products form and move to the narrow and long interstitial space between the seven wires (see Fig. 5). Hence, in PS strands, the initial corrosion products do not exert pressure onto the surrounding porous concrete until the interstitial space is well filled. - unlike the solid rebar. Fig. 5 shows the localized corrosion damage occurred in the PS strand in a long-term corrosion study using ASTM G109 (2009) type specimens (Rengaraju *et al.* 2018). Although no signs of corrosion was visible at the surface of the concrete, significant crevice corrosion was observed on the embedded specimens (observed after autopsying the specimens and cleaning as per ASTM G1 procedure (1990). It was observed that the corrosion products accumulated in the interstitial space between the seven wires.

Figure 7. shows the cross section of a corroded PS strand showing corrosion in the interstitial space and section loss of the outer wires. No signs of visible corrosion stains were observed at the surface of the ASTM G109 specimen from which this corroded strand was extracted.

It is clear that significant corrosion has to occur before the corrosion products can exert pressure and flow through the porous concrete cover, and then reach the surface – manifested by rust stains. Unlike the case of conventionally reinforced structures, visual observations will not help in detecting corrosion damage in a PTC structure, especially at the early periods of corrosion.

Fig. 7. Cross section of PS strand showing section loss of the outer wire and the at the interstitial spaces (Adapted from Rengaraju *et al.* 2017)

At any cross section of a strand, at a given time, only one outermost wire near the exposed surface is assumed to corrode, especially at the early stages of corrosion. Assuming six times volumetric expansion (from steel to rust), it can be said that the cross-sectional area of the corroding sector of the outer wire is equal to that of one of the six interstitial spaces between the seven wires. Using this assumption and Faraday's Law, the time taken to fill the interstitial space, *T*, can be estimated using Faraday's law as follows.

$$T = \frac{26545\, n\, W}{i_{corr}\, W_{eq}\, A_{exp}} \quad (3)$$

where, n is the valency (= 2); W is the mass loss (g); i_{corr} is the corrosion current density of prestressed steel strand (μA/cm^2); W_{eq} is the equivalent weight of steel (g); A_{exp} is the exposed surface area (cm^2). Here, i_{corr} is taken as 0.32 μA/cm^2 (adapted from another ongoing work in IIT Madras). In this manner, if the total duration of wet period in a year is 3 months, then T is estimated to be 30 months of wet period, which is equivalent to 10 years. Hence, the time taken for visible stains to appear in the concrete surface can be more than 10 years from the time of corrosion initiation. This is because, in the early stages, the rust products will fill the interstitial spaces between the wires; then they have to exert pressure and flow radially outward forming rust stains. The above estimate indicate 5% loss of the cross-section of strand by the time rust stains are visible on the concrete surface. Hence, it is not recommended to rely only on visual observation while inspecting PTC systems. It is recommended to inspect and collect data on the chloride profile every 10 years, update the residual service life, implement preventive maintenance measures, and extend the service life by the maximum possible duration with minimal economic investment.

6.0 SUMMARY AND CONCLUSIONS

This study aimed at estimation of the residual service life (RSL) of a coastal bridge in India. A six-year old PTC bridge girder was chosen. D_{Cl} and Cl_s were determined from core samples obtained from the girder. Cl_{th} was estimated for a similar steel-cementitious system from a laboratory study based on linear polarization resistance technique. Then, the RSL of the PTC girder was determined as 42 years based on Fick's 2nd law of diffusion. However, there are chances of initiation even before 42 years. In that case, there will be no visual signs of corrosion till the loss of cross section of around 5%.

The following conclusions are made.

1) Visual observations should not be treated as the primary corrosion detection method for pre-tensioned concrete structures.
2) Periodic field inspection and testing (say, at least 10 years) and updating of residual service life estimates are necessary to assess the condition of the structures
3) Preventive maintenance should be carried out based on the updated residual service life estimate.

Acknowledgement

The authors acknowledge the financial assistance provided by the Ministry of Human Resources Development (MHRD), Government of India through the Indian Institute of Technology (IIT) Madras. The technical support provided by Mr. Sundar Rathnarajan and Ms. Resmi Giriraju is also highly appreciated.

References

ACI 318, 1995. Building Code Requirements for Structural Concrete. American Concrete Institute, Farmington Hills, IL, USA.

Armaghani, J. M., Larsen, T. J., Romano, D. C., 1992. Aspects of Concrete Strength and Durability. Transport Res Rec 1992(1335), pp. 63-9.

ASTM G1, 1990. Standard Practice for Preparing, Cleaning, and Evaluating Corrosion Test Specimens. American Society of Testing and Materials, Conshohoken, PA, USA.

ASTM G109. 2007. Standard Test Method for Determining Effects of Chemical Admixtures on Corrosion of Embedded Steel Reinforcement in Concrete Exposed to Chloride Environments, American Society of Testing and Materials, Conshohoken, PA, USA.

ASTM C1556, 2016. Standard Test Method for Determining the Apparent Chloride Diffusion Coefficient of Cementitious Mixtures by Bulk Diffusion. American Society of Testing and Materials, Conshohoken, PA, USA.

Azuma Y., Miyazato, S., Niitani, K., Yamada, K., and Tokumitu, S. (2007). “Influence of Chloride Ion Content and Stress on Steel Corrosion in Cement Paste Grout.” The 32nd Conference on Our World in Concrete & Structures, Article ID 100032017, Singapore.

Bruce, S.M., McCarten, P.S., Freitag, S.A. and Hasson, L.M. (2008). New Zealand Research Report 337. 72 pp.

FHWA-HRT-12-067, 2012. Summary Report on Literature Review of Chloride Threshold Values for Grouted Post-Tensioned Tendons.

Gettu R., Pillai R.G., Santhanam M., Rathnarajan S., Basavaraj A.S., Rengaraju S., Dhandapani Y. (2018) “Service Life and Life-Cycle Assessment of Reinforced Concrete with Fly ash and Limestone Calcined Clay Cement”, International Conference on Durability of Concrete Structures, U.K. (Submitted)

Karuppanasamy. J, Pillai, R.G., 2017. Statistical Distributions for the Corrosion Rates of Conventional and Prestressing Steel Reinforcement Embedded in Chloride Contaminated Mortar, NACE CORROSION,73(9), pp.1119-1131.

Lau, K. and Lasa, I., 2016. Corrosion of Prestress and Post-tension Reinforced Concrete Bridges. Corrosion of Steel in Concrete Structures, ISBN: 978-1-78242-381-2, pp. 269-288.

Novokshchenov, V. (1989). Salt Penetration and Corrosion in Prestressed Concrete Members. Publication No. FHWA-RD-88-269, Federal Highway Administration, McLean, Va.

NT BUILD 492, 1999. Chloride Migration Coefficient from Non-Steady-State Migration Experiments, NORDTEST.

NZTA report 502, 2003. Assessing pre-tensioned reinforcement corrosion within the New Zealand concrete bridgestock.

Pillai, R. G. (2009). Electrochemical characterization and time-variant structural reliability assessment of post-tensioned, segmental concrete bridges. PhD Thesis, Texas A&M University, College Station, Texas, USA.

Pfeifer, D.W., Landgren, J.R., and Zoob, A. (1987). Protective Systems for New Prestressed and Substructure Concrete, Report No. FHWA-RD-86-193, Federal Highway Administration, McLean, VA.

Polder, R. B., Peelan and Courage M. G. (2012), Non-traditional assessment and maintenance methods for aging concrete structures – technical and non-technical issues, Materials and Corrosion, 63(12), pp. 1147-1153.

Rengaraju S., Godara A., Alapati P., and Pillai, R.G., Macrocell corrosion mechanisms and challenges in testing of steel rebars and prestressing strands in highly resistive concretes, NACE Corrosion Journal, (Submitted, 2018).

SHRP, 1992. Standard Test Method for Chloride Content in Concrete Using the Specific Ion Probe., In Condition Evaluation of Concrete Bridges Relative to Reinforcement Corrosion-Volume 8: Procedure Manual, SHRP-S/FR-92-110, Strategic Highway Research Program, Washington, DC, USA, pp. 85-105, SHRP S330.

Sly, C. (2001). An Initial Look at the Lowe’s Motor Speedway Pedestrian Bridge Collapse, Practical Failure Analysis, 1(2), pp. 7-9.

Stark, D. (1984). “Determination of Permissible Chloride Levels in Prestressed Concrete.” PCI Journal, July–August, Precast/Prestressed Concrete Institute, Chicago, IL.

Trejo, D., Pillai, R.G., Hueste, M.B., Reinschmidt, K., and Gardoni, P. (2009). “Parameters Influencing Corrosion and Tendon Capacity of Post-Tensioning Strands,” ACI Materials Journal, March–April, pp. 144–153.

Wang, H.B., Sagues, A.A., and Powers, R.G. (2005). “Corrosion of the StrandAnchorage System in Post-Tensioned Grouted Assemblies.” NACE International Corrosion Conference Paper No. 05266, Houston TX USA.