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Investigation for Prediction of Time from Onset of Corrosion To Corrosion Cracking

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Abstract: — **Corrosion rate is one of the most important input parameter in corrosion-induced damage prediction models for reinforced concrete (RC) structures. Prediction of time of corrosion initiation to corrosion cracking is crucial factor for evaluation of structural durability. Its accurate assessment and prediction is therefore required if the damage prediction models are to be reliably used to predict both the rate and severity of damage and to plan for maintenance of the structures. To cater with this issue, researchers developed the numerical models for prediction of time of corrosion initiation to corrosion cracking. These models are relatively new and needs further investigation for checking of their global applicability. Thus, present research work aims to carry out comparative study for prediction of time from onset of corrosion to corrosion cracking through numerical & experimental investigation.**

Keywords— **Reinforced Concrete, Corrosion, Steel Rebar, Concrete Cracking, Numerical Model**

I. INTRODUCTION

Corrosion is an electrochemical process, which converts a refined metal to a more stable form, such as its oxide, hydroxide, or sulfide. It is the gradual destruction of metals by chemical reaction with their environment. Passivation refers to the spontaneous formation of an ultrathin film of corrosion products, known as a passive film, on the metal's surface that act as a barrier to further oxidation. Once the passive film breaks down corrosion will start. Corrosion of the reinforcement is one of the major reasons for deterioration of reinforced concrete structures. Corrosion leads to cover cracking, loss of steel cross-section area, loss of stiffness, delaminating of concrete cover, loss of concrete-steel bond strength, and severely reduces the load carrying capacity of RC structures .[7]

A theoretical model of service life for concrete structures with regard to reinforcement corrosion is established by Tuutti [1]. This model shows two stages of deterioration which is caused by corrosion initiation and propagation. First is the initiation period, which represents the time required for CO_2 or Cl ions to diffuse to the steel-to-concrete interface and activate corrosion. Second is the propagation period, which represents the time between corrosion initiation and corrosion cracking.

The researcher [2] put forwarded the research and concluded that Tutti's model does not gives same results compared with times obtained from field and laboratory observations.

Weyers [2] modified the service life model proposed by Thutti [1] further in which the propagation period, is divided into two periods. First one is the free expansion period which considers the time required for corrosion products to fill the porous zone around the corroding steel reinforcing bar. Second period includes the time in which the stress developed, as corrosion products, having filled the porous zone, exert an expansive pressure on the surrounding concrete.

Bazant [3] suggested a mathematical model for RC bridge decks. According to this model time from corrosion initiation to corrosion cracking is mainly dependent on corrosion rate, cover depth, spacing between steel reinforcing bars, diameter of the steel reinforcing bar, and properties of concrete such as tensile strength, modulus of elasticity, Poisson's ratio, and creep coefficient. Assumption of this model that all corrosion products create pressure on the surrounding concrete, underestimates the time to corrosion cracking.

Liu and Weyer's [4] put forwarded Bazant's [3] work further. The model consider same parameters used in Bazant's model but it takes into account the time required for corrosion products to fill a porous zone around the steel reinforcing bar before creating an internal pressure on the surrounding concrete. The time from corrosion initiation to corrosion cracking based on the amount of corrosion products required to cause cracking of concrete cover.

Tamer El Maaddawy and Khaled Soudki's [5] suggested a mathematical model which explains relationship between the steel mass loss and the internal radial pressure caused by corrosion. The ingress of corrosion products into the open radial cracks was ignored in this model. Thus the model was further modified by C.H. Lu *et.al.* [6]

C.H. Lu *et. al.* [6] contributes towards a quantitative relationship between the amount of steel corrosion and concrete cover cracking, which takes the amount of corrosion products accommodated within the radial cracks into account.

Based on above literature survey it is found that there is a need for checking the accuracy of the numerical model through comparison of the model's predictions with experimental data. Hence the aim of present research work is: Comparative study for prediction of time from onset of corrosion to corrosion cracking through numerical & experimental investigation.

II. METHODOLOGY

A. *Numerical investigation*

In numerical investigation the time from corrosion initiation to corrosion cracking is calculated by two formulas in literature. These formulas accounts for the time required for corrosion products to fill a porous zone before they start inducing expansive pressure on the concrete surrounding the steel reinforcing bar. [5][6]

1. *Formula 1*

Formula given by Tamer El Maaddawy and Khaled Soudki's [5] considers that there is a porous zone around the steel reinforcing bar. The four basic assumptions consider for the model are,

- The corrosion products are formed uniformly around the steel reinforcing bar which results in a uniform expansive stresses around the steel bar.
- The volume expansion caused by corrosion creates strain only in concrete (i.e. strain in steel is neglected).
- Mechanical and material properties of corrosion products were complicated neglected.
- The ingress of corrosion products into the open radial cracks ignored.

$$T_{cr} = \left[\frac{7117.5(D+2\delta_0)(1+\upsilon+\psi)}{iE_{ef}}\right]\left[\frac{2Cf_{ct}}{D}+\frac{2\delta_0 E_{ef}}{(1+\upsilon+\psi)(D+2\delta_0)}\right] \qquad (1)$$

Where,

T_{cr} = the time from corrosion initiation to corrosion cracking.

D =diameter of bar

δ_0 = thickness of porous zone.

μ = Poisson's ratio of cylinder

Ψ = D'2/2C (C+ D'), D' = d+2 δ0

C = wall thickness of cylinder.

i = current density

f_{ct} = tensile strength of concrete

E_{ef} = effective elastic modulus of concrete

= $[E_c / (1 + \phi_{cr})]$,

E_c is the elastic modulus of concrete,

ϕ_{cr} is the concrete creep coefficient.

2. *Formula 2*

Formula given by C.H. Lu *et.al.* [6] considers the penetration of corrosion products into radial corrosion cracks. . The four basic assumptions consider for the model are,

- Corrosion process is spatially uniform around the steel reinforcement which results in a uniform radial expansive pressure at the steel-concrete interface,
- The concrete around the steel reinforcing bar is modeled as a thick-walled cylinder and the wall thickness equals to the thinnest concrete cover,
- The stresses in concrete and reinforcement are induced only by the expansion of corrosion products, and
- The radial cracks will develop from the cylinder's inner surface to outside, and the corrosion products shall be accommodated in these cracks.

$$t_{cr} = t_1 + t_2 = \left(1+k\frac{c}{d}\right)t_1 = 234762(d+kc)\times \frac{\left\{\left(0.3+0.6\frac{c}{d}\right)\cdot\frac{f_{ct}}{E_{cef}}\left[\frac{(r_0+c)^2+r_0{}^2}{(r_0+c)^2-r_0{}^2}+\upsilon_c\right]+1+\frac{2\delta_0}{d}\right\}^2-1}{(n-1)\cdot i_{corr}} \qquad (2)$$

Where,

t_{cr} = the time from corrosion initiation to corrosion cracking.

C = bottom clear cover.

d =diameter of bar

f_{ct} = tensile strength of concrete.

E_{cef} = effective modulus of elasticity of concrete

= $[E_c / (1+ \phi_{cr})]$

E_c is the elastic modulus of concrete,

ϕ_{cr} is the concrete creep coefficient

r_0 = radius of bar + thickness of porous zone

δ_0 = thickness of porous zone.

μ_c = Poisson's ratio of concrete

η = ratio of volume expansion of corrosion product to volume of iron consumed.

i_{corr} = current density.

k = Modified coefficient

= 0.15-0.30 (For accelerated corrosion)

= 0.8-1.0 (For natural corrosion)

From above discussion it is clear that Formula 1 completely ignores the material properties of concrete and mechanical properties of corrosion as well as neglects the ingress of corrosion products into the open radial cracks. Whereas Formula 2 takes into the account of amount of corrosion products accommodated within the radial cracks. Hence the present study is focused on applicability of Formula 2 for practical use.

In present paper, for numerical investigation the Poisson's ratio of concrete (μ_c) was considered as 0.2. The thickness of the porous zone (δ_0) is typically in the range of 10~20μ, the mean value of 15μm was adopted [6]. The ratio of volume expansion of corrosion products n is generally between 2 to 4 [6], and n=2.5~3.0 is practical for normal corrosion of steel bars. In current study n=2.7 was considered. IS 456-2000 suggested methods were used to calculate tensile strength and modulus of elasticity of concrete For M35 grade.

B. *Experimental details*

- Material Properties and specimen specifications.

For experimental investigation cylindrical RC specimens having size 60mm x 110mm as shown in Fig. 1 were cast. TMT 500 grade reinforcing bar having length 110mm and 16mm diameter was used as a concentric reinforcement in the cylindrical specimens.

Ordinary Portland cement of strength 53 MPa was used for preparation of concrete mixes of M35 grade. Natural river sand confirming to zone I as per IS 383-1970 was used as fine aggregates and crushed stone of nominal size 20 mm was used as coarse aggregates. As per IS 10262:2009 the concrete mix design was done and the mix proportion obtained was 1:1.65:2.874 with water–cement ratio of 0.5. For electrical connections, the steel bar was drilled and threaded at one end to accommodate the threaded copper screw. Then epoxy resin was applied for the length of 60 mm from top and 10 mm form bottom to protect this portion from the corrosion activity. The remaining middle portion of 40 mm was subjected to accelerated corrosion process. The epoxy resin was allowed to harden for 24 h.

For casting of specimens, a special moulding system was used as shown in Fig. 2. The specimens were cast in inverted position to maintain the bottom cover and concentric position of rebar.

Demoulding of the cylinders was done after 24 h of casting. Then the specimen kept for curing for the period of 7-days at a room temperature and relative humidity of 100%. On 8th day the specimens were immersed in 5% NaCl solution, at a room temperature for 24 h to ensure full saturation of the test specimen. From ninth day constant potential was applied to the specimen to accelerate the corrosion process using impressed current technique.

Fig 1: RC Specimen with concentrated reinforcement

Fig 2: Casting setup - Specimen's stand

C. *Application of current*

The specimens were subjected to accelerated corrosion using impressed current technique.

A stainless steel (SS) mesh in the form of a strip was used as cathode. The reinforcing steel bar was connected to the positive terminal of the external DC source and negative terminal was connected to the SS mesh. It is more common to maintain a constant voltage between the cathode and the anode and hence a constant voltage of 3 V was applied between steel rebar and SS mesh. Anode to cathode current corresponding to constant applied voltage was monitored every day. The testing was stopped when the crack due to corrosion appeared and become distinct on the surface of concrete specimen.

D. *Half-cell Potential Technique*

Several methods are used for the diagnosis, detection and measurement of corrosion of RC specimens reinforcing steel. The practically widely used method for detection of corrosion is the half-cell potential (HCP) method. The behavior of the steel under corrosion is characterized by measuring its half-cell potential.

HCP consist of an electrode which forms one half of the cell and the reinforcing steels in the concrete the other. Saturated calomel electrode (SCE) was used as reference electrode in present work.

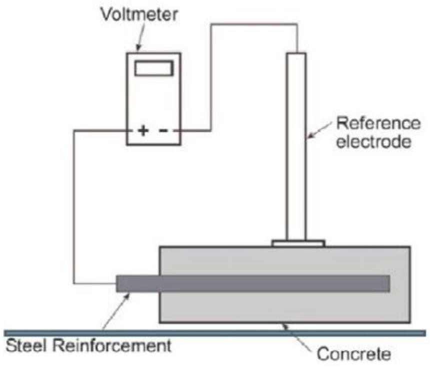

Fig 3: Half Cell Potential (HCP) [7]

Table I: Corrosion Condition Related With HCP Measurement [7]

Open circuit potential values for SCE (mV)	Corrosion condition
< -426	Severe corrosion
< -276	High (<90% risk of corrosion)
-126 to -275	Intermediate corrosion risk
> -125	Low(10% risk of corrosion

III. RESULTS AND DISCUSSION

For experimental investigation variation of HCP with time was plotted for three specimens.

For first specimen (Fig. 4) it can be observed that, on the second day of testing itself, and for second (Fig. 5) and third (Fig. 6) specimen on the first day of testing itself, the HCP value observed was more negative than -276 mV indicating onset of corrosion. The potential value continuously dropped with the progress of time. On 6th day the potential value found to be more negative than -426 mV indicating sever corrosion. For the all specimen showed presence of hair line crack on the surface of concrete on 9th day.

Thus, experimentally based on HCP measurements and visual observations the time required from onset of corrosion to cracking of concrete was calculated as 7 days for first specimen and 8days for second and third specimen.

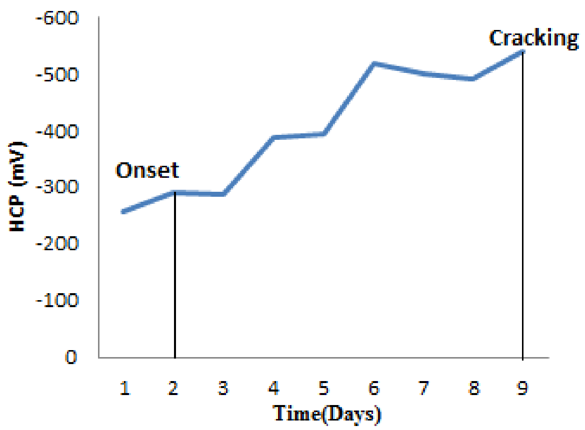

Fig 4: Variation Half Cell Potential with time for specimen 1.

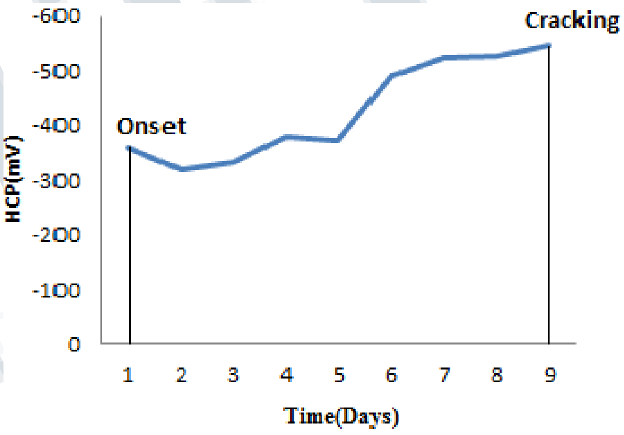

Fig 5: Variation Half Cell Potential with time for specimen 2.

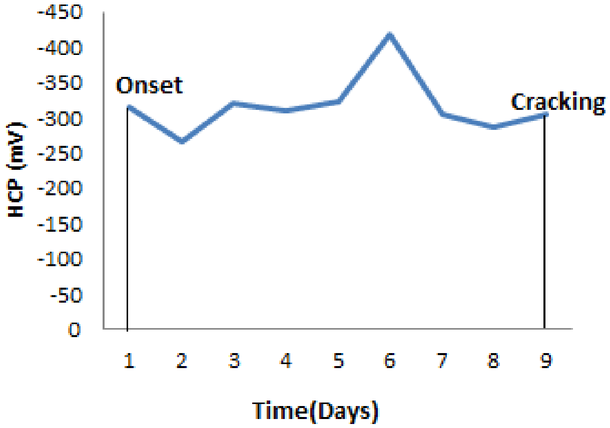

Fig 6: Variation Half Cell Potential with time for specimen 3.

Numerical investigation was done by using Formula 2. The comparisons of numerical values with experimental results are shown in TABLE II.

Table II Experimental And Numerical Time

	Experimental values (hrs)	Numerical values (hrs)
Specimen 1	168	26
Specimen 2	192	50
Specimen 3	192	95

From TABLE II it is clear that there is much difference in time from onset of corrosion to corrosion cracking. This may be due to difference in assumptions used for numerical calculations and experimental results.

IV. CONCLUSION

The present paper investigates study of various formulas available to calculate time from onset of corrosion to corrosion cracking and experimental calculation of time required from onset of corrosion to cracking of concrete.

- ❖ For accurate calculation of time from onset of corrosion to corrosion cracking, the actual values of various parameters used in numerical analysis such as poisson's ratio, creep coefficient, modulus of elasticity etc. are required to be derived experimentally.
- ❖ As the presence of micro cracks inside concrete cannot be observed externally. There is a need for suitable nondestructive technique capable of identifying development of micro cracks.

REFERENCES

[1] Tuutti K. ,"Service life of structures with regard to corrosion of embedded steel". Performance of concrete in marine environment, ACI SP-65. Detroit, Michigan: American Concrete Institute 223–36, 1980.

[2] Weyers RE. ,"Service life model for concrete structures in chlorideladen environments" . ACI Mater J 1998; 95 (4):445–53, 1998.

[3] Bazant, Z. P., "Physical model for steel corrosion in concrete sea structures-application". Journal of the Structural Division 105(6): 1155-1166, 1979..

[4] Liu, Y.P. & Weyers, R.E., "Modeling the time to corrosion cracking in chloride contaminated reinforced concrete structures". ACI Materials Journal 95(6): 675-681, 1998.

[5] Maaddawy, T.E. & Soudki, K. ,"A model for prediction of time from corrosion initiation to corrosion cracking". Cement and Concrete Composites 29 (3): 168-175, 2007.

[6] C.H. Lu, R.G. Liu & W.L. Jin, "A model for predicting time to corrosion-induced cover cracking in reinforced concrete structures". Fracture Mechanics of Concrete and Concrete Structures - Assessment, Durability, Monitoring and Retrofitting of Concrete Structures- B. H. Oh, et al. (eds) 2010 Korea Concrete Institute, Seoul, ISBN 978-89-5708-181-5, 2010.

[7] ASTM Standard test method for half cell potentials of uncoated reinforcing steel in concrete. C876-91,1999.

[8] M. S. Shetty "concrete technology" (2012).

[9] A.R. Santhakumar "Concrete technology"15th impression 2014.

[10] IS 516 – 1959 (Reaffirmed 1999). Methods of tests for strength of concrete. New Delhi: Bureau of Indian Standards.

[11] IS 383-1970 (Reaffirmed 2002). Specification for coarse and fine aggregates from natural sources for concrete. New Delhi: Bureau of Indian Standards.

[12] IS 10262:2009. Concrete mix proportioning – guidelines. New Delhi: Bureau of Indian Standards.

[13] IS 456:2000. Plain and reinforced concrete-code of practice. New Delhi: Bureau of Indian Standards.