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PREDICTION OF CORROSION-FREE SERVICE LIFE OF CONCRETE STRUCTURES ALONG COASTAL REGIONS: A NUMERICAL FRAMEWORK

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ABSTRACT

Chloride-induced rebar corrosion in concrete along coastal regions is currently a serious problem in the world. To estimate the corrosion-free service life of concrete structures, this study proposes an effective numerical framework through a combination of finite element and finite difference schemes. By considering the presence of rebars in concrete and by incorporating enhanced parameter prediction models, this framework solves the associated mass transport partial differential equations representing the process of chloride ingress in concrete. It is observed that, in addition to water-binder ratio and cover depth, the sustenance of corrosion-free service life of concrete under chloride environment is significantly governed by rebar size and its locations. Moreover, estimated time-variant reductions in cross-sectional area of longitudinal rebar and flexural capacity of the reinforced concrete beam show a clear change in slope, when the middle bars start corroding.

Keywords: Chloride, concrete, corrosion, finite element method

INTRODUCTION

Chloride-induced rebar corrosion in concrete is currently a serious problem in the world. Based on conservative estimates, one-half of highway bridges are deteriorating due to rebar corrosion in the developed countries, and billions of dollars are required to repair or rehabilitate the damaged structures. Reinforced concrete (RC) structures fail to perform their intended functions, and their service lives are much shorter than what they were designed for. Rebar corrosion is widely accepted as a reasonable explanation for such premature deterioration incurred by RC structures. Moreover, a majority of the world’s population inhabit marine atmosphere (coastal) zones, the interface between the land and seawater. In such zones, salinity is the main source of built infrastructure deterioration; predominantly due to the chloride-induced rebar corrosion of the embedded steel. Chloride ions present in coastal zones, while being assisted by the prevailing wind, temperature and humidity, intrude into the porous concrete. Chloride ingress into the concrete induces rebar corrosion, which is a complex interaction between many physical and chemical processes.

Rebars embedded in concrete are: (i) chemically protected by the highly alkaline passive layer (pH ~ 13–14); and (ii) physically protected by the concrete cover acting as a barrier against the intrusion of aggressive species. However, chloride-induced corrosion begins when the concentration of chloride at the steel bar level reaches a critical chloride content or a chloride threshold value thereby destroying the protective layer. Critical chloride content or chloride
threshold value of rebar in concrete can be defined as the concentration of chloride at the depth of the rebar that is necessary to sustain localized breakdown of its passive protection layer and hence initiate its active corrosion. The time taken by chloride ions from external sources (such as marine environments) to reach a threshold value at the rebar depth is defined as time-to-corrosion initiation (TCI). When corrosion occurs, the nature of the attack can result in an extreme loss of rebar cross-sectional thereby affecting significantly the tensile capacity of the corroded component. In addition, damage associated with corrosion is manifested in the form of cracking and spalling of the concrete cover.

In many countries, including India, coastal zones are not only thickly populated but are also industrialized to a great extent. In such zones, in addition to sea-salt spray, industrial effluents could contribute to the presence of chloride ions (Costa and Vilarrasa, 1993). Industrial atmospheres have been reported to contain considerable amount of chloride (Cl\textsuperscript{−}), sulphur dioxide (SO\textsubscript{2}) and oxides of nitrogen (Natesan et al., 2006). The major part of India is generally subjected to moderate climatic conditions; hence, early deterioration of concrete structures is not a big area of concern. However, concrete structures in coastal and industrial belts and certain extreme climatic zones are subjected to aggressive environment, and hence the problems of early deterioration of concrete are a cause of concern in India. In addition, poor quality of construction and environmental pollution in major cities has also lead to early deterioration of concrete structures located in moderate climate zones and cities, respectively (Kulkarni, 2009). Hence, civil engineering structures like buildings, bridges, docks, harbors located in these zones are highly vulnerable to earlier deterioration due to atmospheric chlorides attack.

Vast majority of existing works in the literature assume a uniform corrosion pattern around the rebar circumference (e.g. Ahmed et al., 2007; Bhargava et al., 2003; Chen and Mahadevan, 2008; Li et al., 2006; Liu and Weyers, 1998; Pantazopoulou and Papoulia, 2001; Val et al., 2009). This is rarely the case in practice as chloride ions generally take a long time to transport to the steel-concrete interface that is facing the interior of concrete because of the impermeability of steel bars. Therefore, steel surface facing the outside environment is usually subjected to a higher-level concentration of chloride ions, and so gets depassivated and starts to corrode earlier than the other side of steel surface does. While on the surface facing the interior of concrete, the amount of chloride ions is maintained at a very low-level because of the impermeable nature of steel bars. Hence, breakdown of surface passive film and corrosion of steel could hardly happen in this area before the cracking of concrete cover. Variation in the onset of corrosion reaction over the entire rebar surface, results in non-uniform distribution of steel corrosion. Hence, the morphology of steel corrosion in concrete is quite different from the widely employed assumption of uniform corrosion, and it will undoubtedly propel the evolution of corrosion-induced damage in a different way than uniform corrosion does (Liu and Li, 2004; Oh and Jang, 2003). In addition, it was reported that the chloride accumulates in front of a rebar, which is much more pronounced for larger-size bars (Oh and Jang, 2003). The higher accumulation of chlorides at bar location causes faster corrosion of rebars. Hence the real state of corrosion is not uniform around a rebar. The importance of considering the real state of corrosion in rebar is also supported by the fact that the effect of non-uniform corrosion induced stresses in RC are more conducive to cover cracking than uniform corrosion (Xia et al., 2012).

This study presents a numerical framework that can efficiently quantify time-variant degradation in the flexural capacity of a corroding RC beam based on non-uniform corrosion. This framework considers the effects of rebar size and location on the process of chloride ingress into concrete and focuses on rebar corrosion due to atmospheric chlorides by
considering diffusion as the dominant mode of chlorides ingress. This work is presented in two parts. The time to first onset of corrosion reaction for the corner and middle bars embedded in a RC beam are presented in the first part of the study for 11 atmospheric exposure stations in India by incorporating various parameters such as water/binder ratio \((w/b)\), concrete cover thickness and rebars size. The second part of the work presents the quantitative estimations of non-uniform reductions in the net cross-sectional areas of the rebars in the RC beam. This part also presents an assessment of the time-variant degradation in the flexural capacity of a corroding RC beam based on non-uniform corrosion.

**CHLORIDE INGRESS MODEL**

The problem of chloride ingress into concrete can effectively be studied as the interaction between three phenomena, namely: heat transport; moisture transport; and chloride transport. Each of these phenomena is represented by a partial differential equation (PDE) and their interaction is considered by solving them simultaneously. The governing PDEs representing the process of chloride ingress into concrete can be expressed in the following general form:

\[
\gamma \Phi + \nabla \cdot (\Theta \nabla \Phi) = 0
\]  

The correspondence between \(\gamma\), \(\Theta\), \(\Phi\) and the terms for the transport quantity is presented in Table 1.

<table>
<thead>
<tr>
<th>Transport</th>
<th>(\Phi)</th>
<th>(\gamma)</th>
<th>(\Theta)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chloride</td>
<td>(C_f)</td>
<td>1</td>
<td>(D_c^a)</td>
</tr>
<tr>
<td>Moisture</td>
<td>(h)</td>
<td>(\partial w_{e} / \partial h)</td>
<td>(D_h)</td>
</tr>
<tr>
<td>Heat</td>
<td>(T)</td>
<td>(\rho c_p)</td>
<td>(D_T)</td>
</tr>
</tbody>
</table>

For chloride transport, \(C_f\) is the free chloride content dissolved in the pore solution (kg/m\(^3\)), \(D_c^a\) and \(D_h^e\) represent apparent chloride and humidity diffusion coefficients (m\(^2\)/s), respectively:

\[
D_c^a = \frac{D_{c,ref} f_c(T) f_c(t) f_c(h)}{1 + \frac{1}{w_e} \left( \frac{\partial C_f}{\partial C_f} \right)}
\]

\[
D_h^e = \frac{D_{h,ref} f_h(T) f_h(h) f_h(t_e)}{1 + \frac{1}{w_e} \left( \frac{\partial C_f}{\partial C_f} \right)}
\]
where $D_{c,\text{ref}}$ and $D_{h,\text{ref}}$ are reference chloride and humidity diffusion coefficients (m$^2$/s) measured under standard conditions (Saetta et al., 1993), $w_e$ is evaporable water content (m$^3$ of water/m$^3$ of concrete), $f_c$ and $f_h$ are modification factors to account for the effects of temperature, relative humidity, ageing and the level of hydration in concrete. These factors are detailed in Muthulingam and Rao, 2014. The term $\frac{\partial C_f/\partial C_r}$ represents the binding capacity of the cementitious system which is the slope of either Langmuir or Freundlich binding isotherm. The binding isotherm relates the free and bound chloride content at equilibrium and is characteristic of each cementitious system (Tang and Nilsson, 1993).

For moisture transport, $D_h$ represents humidity diffusion coefficient (m$^2$/s) and the derivative of water content with respect to pore relative humidity (i.e. $\frac{\partial w_e}{\partial h}$) is defined as moisture capacity (m$^3$ of water/m$^3$ of concrete). At standard temperature and pressure, adsorption isotherm relates evaporable water content and pore relative humidity. Based on thermodynamic principle of adsorption, Braunaer-Skalny-Bodor model considers this relationship to depend on temperature, water-to-binder ratio (a ratio by mass) and level of hydration in concrete, $t_e$ in days (Xi et al., 1994). From the adsorption isotherm, moisture capacity can be obtained by taking its derivative with respect to $h$:

$$\frac{\partial w_e}{\partial h} = \frac{CkV_m(Chk^2 - h^2k^2 + 1)}{(hk - 1)^2 (Chk - hk + 1)^2} \text{[m}^3\text{of water/m}^3\text{of concrete]} \quad (4)$$

where $C$ is a constant that takes into account the influence of change in temperature on the adsorption isotherm, $k$ is a constant resulted from the assumption that the number of adsorbed layers is a finite small number and $V_m$ represents the monolayer capacity. These parameters are detailed in Muthulingam and Rao, 2014.

For heat transport, $T$ is temperature (K), $\rho$ is the density of concrete (or) rebar (kg/m$^3$), $c_p$ is the specific heat capacity of concrete (or) rebar (J/kg K), and $D_T$ is the thermal conductivity of concrete (or) rebar (W/m K).

## RATE OF CORROSION

The corrosion rate ($i_{corr}$) of rebars in concrete structures is one of the most important parameters for making reasonable service life prediction (Li et al., 2006; Otieno et al., 2012). Corrosion rate of rebars in concrete structures is affected by various factors, namely, supplementary cementitious materials, moisture content, cyclic wetting and drying, sustained loading, loading history, concrete resistivity, concrete quality, cover depth, temperature, cracking, dissolved oxygen content and exposure conditions (Otieno et al., 2012). By considering few or more of the above-mentioned factors, various corrosion rate prediction models were proposed in literature. Corrosion level of the rebar is generally determined in terms of depth of the attack penetration (e.g. Val et al., 2009). The corrosion penetration depth, $p_d$ (mm) at an exposure time, $t$ (years) after corrosion initiation, $t_e$ can be estimated based on Faraday’s law (Rodriguez et al., 1996):
Vu and Stewart (2000) expressed the corrosion rate up to one year ($i_{\text{corr}(0)}$, $\mu$A/cm$^2$) after the end of the corrosion initiation phase for an ambient relative humidity of 75% and a temperature of 20 °C as given in Eq. (6).

$$i_{\text{corr}(0)} = \frac{37.8 \left[ 1 - \left( \frac{w}{b} \right) \right]^{-1.64}}{C_c} \quad [\mu$A/cm$^2]$$  

where $C_c$ represent concrete cover thickness (cm) and $w/b$ represents water-to-binder ratio.

During the corrosion propagation phase $i_{\text{corr}}$ can be expressed as follows:

$$i_{\text{corr}} = i_{\text{corr}(0)} 0.85(t - t_c)^{0.29} \quad [\mu$A/cm$^2]$$  

where $t$ is the time to which $i_{\text{corr}}$ is to be predicted (years) and $t_c$ is time-to-corrosion initiation (years). For the corrosion rate model of Vu and Stewart (2000), $p_d(t)$ can be expresses as:

$$p_d(t) = 0.5249 \left[ 1 - \left( \frac{w}{b} \right) \right]^{-1.64} (t - t_c)^{0.71}$$  

Further, BS 6349–1 suggested the mean and upper limit values of corrosion rate for various exposure zones that apply to each face exposed to the environment of the zone.

**FLEXURAL CAPACITY OF RC BEAM**

The time-variant flexural capacity of the corroding RC beam ($M_{ur}$) due to the net reduction in the cross-sectional areas of the longitudinal rebars can be given by Eq. (9).

$$M_{ur}(t) = f_s A_s(t) \left( 1 - z_2 \frac{f_{c} A_i(t)}{z_1 z_3 f_{c} b d_c} \right) d_c$$  

where $A_i$ is the area of the longitudinal tensile rebar, $f_s$ is the yield strength of the steel, $d_c$ is the effective depth of the rebar, $f_{c}$ is the compressive strength of concrete and $b$ is the width of the section. The value of $z_1$ (conversion factor for compressive stress block of concrete) for a parabolic-rectangular stress block is 0.81. The value of $z_2$ (ratio of the depth of centroid of the stress block to the depth of neutral axis) for a parabolic-rectangular stress block is 0.42. The value of $z_3$ (ratio of the compressive strength of concrete in the girder to that of the design cube compressive strength of concrete) was taken as 0.67, which is based on the size factor equal to 0.85 while using cylinder compressive strength, and the conversion of cylinder strength to cube strength (Pillai and Menon, 2009).
When RC beams undergo severe corrosion, other structural damage mechanisms, namely, concrete cracking, delamination and spalling of the concrete cover, reduction of concrete cross-section and loss of bond between the rebar and concrete are also observed (Bhargava et al., 2011). However, robust prediction models that can estimate the flexural capacity of RC beams by incorporating the damage mechanisms are hardly found in the literature (Stewart and Suo, 2009). Therefore, in the current study, flexural capacity of the corroding RC beam is evaluated without considering the related structural damage mechanisms.

**ATMOSPHERIC CHLORIDES—INDIAN SCENARIO**

The chloride deposition rate is a very important parameter for evaluating the corrosion of metals. Estimating the amount of chlorides present in the atmosphere through deposition measurements by using either wet or lead peroxide candle, is a part of standardized procedures to measure the amount of chloride salts that are captured from the atmosphere on the exposed area of the apparatus. Recently, the chloride deposition rate (mg/m$^2$/day) values have been reported for 11 exposure stations in India by a study conducted to evaluate atmospheric corrosion of metals at these stations (Natesan and Palaniswamy, 2009; Natesan et al., 2008). These stations have different temperature, humidity and pollutant (both liquid and gas) concentrations. Fig. 1 shows the location of 11 stations.

![Fig. 1 - Atmospheric exposure stations in India](image)

Furthermore, 11 atmospheric exposure stations are categorized into five types based on the type of atmosphere prevailing at a particular station, viz. marine, industrial, marine-industrial, rural and urban stations (Natesan and Palaniswamy, 2009; Natesan et al., 2008). The stations 2, 3, 4, 7, 8 and 9 are categorized as marine stations while station 10 is categorized as the industrial station. Further, the stations 1 and 5 are categorized as marine-industrial and hill stations respectively. The stations 6 and 11 are categorized as rural and urban station respectively.

Although chloride deposition rate is a very important parameter for evaluating the corrosion of metals, in the case of RC structure, it is not directly related to corrosion of the rebar (Tanaka et al., 2001). The cumulative chloride deposition on the surface of the measuring
apparatus is correlated to the chlorides accumulation on the surface of the concrete through empirical relationships. One such empirical relationship proposed for correlation is proposed by Tanaka et al. (2001), and the same is given in Eq. (10).

\[ C_s = 1.5 C_d^{0.4} \text{ (kg/m}^3) \]  

(10)

where, \( C_d \) represents chloride deposition rate (mg/dm\(^2\)/day). The chloride deposition rate and the equivalent surface chloride content (\( C_s \)) for 11 atmospheric exposure stations along with their annual mean temperature and relative humidity are listed in Table 2. It can be observed from Table 2 that among 11 atmospheric exposure stations, two of the stations, namely, Chennai and MPT have relatively higher surface chloride contents.

Table 2 - Data for 11 atmospheric exposure stations in India

<table>
<thead>
<tr>
<th>Station name</th>
<th>Mean temperature (°K)</th>
<th>Mean humidity</th>
<th>relative humidity</th>
<th>( C_d ) (mg/m(^2)/day)</th>
<th>( C_s ) (kg/m(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Min.</td>
<td>Max.</td>
<td>Min.</td>
<td>Max.</td>
<td>75.21</td>
</tr>
<tr>
<td>Manali</td>
<td>292</td>
<td>312</td>
<td>0.61</td>
<td>0.86</td>
<td>75.21</td>
</tr>
<tr>
<td>Chennai</td>
<td>293</td>
<td>312</td>
<td>0.59</td>
<td>0.95</td>
<td>304.60</td>
</tr>
<tr>
<td>Cuddalore</td>
<td>293</td>
<td>308</td>
<td>0.64</td>
<td>0.90</td>
<td>39.49</td>
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<tr>
<td>Nagapattinam</td>
<td>295</td>
<td>307</td>
<td>0.61</td>
<td>0.80</td>
<td>18.18</td>
</tr>
<tr>
<td>Tuticorin</td>
<td>293</td>
<td>307</td>
<td>0.58</td>
<td>0.95</td>
<td>47.65</td>
</tr>
<tr>
<td>Mahendragiri</td>
<td>292</td>
<td>310</td>
<td>0.50</td>
<td>0.95</td>
<td>Nil</td>
</tr>
<tr>
<td>Mangalore</td>
<td>292</td>
<td>307</td>
<td>0.63</td>
<td>0.92</td>
<td>69.00</td>
</tr>
<tr>
<td>MPT</td>
<td>290</td>
<td>306</td>
<td>0.61</td>
<td>0.86</td>
<td>266.40</td>
</tr>
<tr>
<td>NIO, Goa</td>
<td>289</td>
<td>304</td>
<td>0.63</td>
<td>0.89</td>
<td>Nil</td>
</tr>
<tr>
<td>Mumbai</td>
<td>292</td>
<td>306</td>
<td>0.56</td>
<td>0.94</td>
<td>30.00</td>
</tr>
<tr>
<td>Surat</td>
<td>288</td>
<td>313</td>
<td>0.62</td>
<td>0.89</td>
<td>13.80</td>
</tr>
</tbody>
</table>

**PROBLEM STATEMENT**

For further study, a RC beam section of size 350 × 700 mm shown in Fig. 8.4(a), reported in the literature (Pillai and Menon, 2009) is considered. In the considered cross-section; (i) \( C, d \) and \( d_e \) represent cover thickness, rebar diameter size and effective depth, respectively; and (ii) four rebars are present, of which two bars of interest in future discussion are named B\(_1\) (middle bar) and B\(_2\) (corner bar). Moreover, to consider the effects of rebar sizes and locations on the spatial and temporal evolution of total chloride content (TCC) and TCI around the steel-concrete interface, various bar diameter (12, 16, 20, and 25 mm), cover thickness (25–75 mm in multiples of 5 mm) and \( w/b \) ratios (0.4, 0.5, and 0.6) are also considered. The adopted RC cross-section is discretized using four noded isoparametric quadrilateral elements as shown in Fig. 2. Care is taken to ensure that sufficient number of nodes is present along the circumference of the rebar (40 nodes) to capture the smooth spatial progress of TCC and TCI.
The external environmental conditions that are specific to each of the atmospheric station need to be imposed on the exposed boundaries of the numerical model. The current study considers that, all the four sides of the RC beam are exposed to atmospheric chlorides. Using the mean temperature and relative humidity distribution data listed in Table 8.1, and assuming a sinusoidal variation similar to that reported in the literature (Bastidas-Arteaga et al., 2011), the monthly variations of temperature and humidity are obtained by curve fitting techniques as a function of time expressed in years:

\[
T_{\text{env}}(t) = \frac{T_{\text{max}} + T_{\text{min}}}{2} + \frac{T_{\text{max}} - T_{\text{min}}}{2} \sin(2\pi t)
\]  

(11)

\[
h_{\text{env}}(t) = \frac{h_{\text{max}} + h_{\text{min}}}{2} + \frac{h_{\text{max}} - h_{\text{min}}}{2} \sin(2\pi (t - 0.5))
\]  

(12)

In FE analysis, the material properties of rebar, namely, density, specific heat capacity and thermal conductivity of rebar are taken as 7850 kg/m$^3$, 620 J/kg.°K and 50 W/m.°K, respectively. Moreover, $D_{h,\text{ref}}$ values of $1 \times 10^{-12}$, $5 \times 10^{-12}$ and $2.5 \times 10^{-11}$ m$^2$/s are considered (Saetta et al., 1993) for w/b ratios of 0.4, 0.5, and 0.6, respectively. In addition, 0.8 kg/m$^3$ (AS 1379:2007) of TCC is considered as critical chloride content.

**SOLUTION PROCEDURE**

a.1. Determine actual temperature profile throughout the FE mesh (including rebar) from by taking into account the initial temperature profile,

a.2. Determine humidity profile throughout the FE mesh (excluding rebar) by taking into account the temperature profile estimated in the previous step and the initial humidity profile,
a.3. Perform iterative procedure to obtain the actual profile of \( h \) by using the profile of \( h \) obtained from the previous iteration until a given convergence criterion is reached,

a.4. Evaluate the amount of evaporable water from adsorption isotherm,

a.5. Evaluate free chloride profile throughout the FE mesh (excluding rebar) by taking into account the temperature and humidity profiles estimated in the previous step and the initial free chloride profile,

a.6. Perform iterative procedure to obtain the actual profile of \( C_f \) by using the profile of \( C_f \) obtained from the previous iteration until a given convergence criterion is reached,

a.7. Determine TCC values, by using the chloride binding relationship and the adsorption isotherm,

a.8. If TCC values at the level of the rebar have reached the critical value, corrosion is assumed to have initiated,

a.9. Determine time-dependent corrosion penetration depth \( p_d(t) \) at depassivated region along the perimeter of the rebar,

a.10. Evaluate non-uniform corrosion states of rebar based on the estimated spatial distributions of \( p_d(t) \),

a.11. Numerically calculate the time-variant net reduction in the cross-sectional area of rebar based on the estimated corrosion penetration depth.

RESULTS AND DISCUSSION
TIME-TO-CORROSION INITIATION

From the numerical analysis, it is observed that the rebars in RC beam that are located in two of the 11 stations, namely, Chennai and MPT experience corrosion over the simulation period of 100 years. This is because; the surface chloride content values reported at these stations are relatively higher. Higher humidity and temperature values at the Chennai station could also be a reason for the corrosion. Table 3 shows the time to first onset of corrosion reaction for \( B_1 \) and \( B_2 \) bars at the Chennai station, for various combinations of rebar size, cover thickness and \( w/b \) ratio. Similarly, Table 4 shows the time to first onset of corrosion reaction for \( B_1 \) and \( B_2 \) bars at MPT station, for various combinations of cover thickness, rebar size and \( w/b \) ratio.

From Tables 3 and 4, it can be observed that, for the given values of cover thickness and water-to-binder ratio, TCI for \( B_2 \) bar is faster than that for \( B_1 \) bar. Additionally, the ratio between TCI of \( B_1 \) and \( B_2 \) bar falls within the range of 1.3–1.6, indicating TCI for \( B_2 \) bar is 30–60% faster than that for \( B_1 \) bar. The direct implication of this observation is that the subsequent onset of cracking, as corrosion level increases over time would be much more serious in concrete cover around the corner bars than that in concrete cover around the middle bars. Therefore, corner bars need more protection from external chloride environment. Between these two stations, TCI values for both \( B_1 \) and \( B_2 \) bars of the RC beam at Chennai station is higher than that at MPT station. This differences in TCI values between them are observed to range between 2–45 years for various combinations of cover thickness, rebar size and \( w/b \) ratio.
Table 3 - TCI values for Chennai station

<table>
<thead>
<tr>
<th>$d$</th>
<th>$C_c$</th>
<th>$B_1$ bar</th>
<th>$B_2$ bar</th>
<th>$d$</th>
<th>$C_c$</th>
<th>$B_1$ bar</th>
<th>$B_2$ bar</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$w/b$ ratio</td>
<td>$w/b$ ratio</td>
<td></td>
<td></td>
<td>$w/b$ ratio</td>
<td>$w/b$ ratio</td>
</tr>
<tr>
<td>25</td>
<td></td>
<td>0.4</td>
<td>0.5</td>
<td>0.6</td>
<td>0.4</td>
<td>0.5</td>
<td>0.6</td>
</tr>
<tr>
<td>30</td>
<td>22.3</td>
<td>8.5</td>
<td>5.4</td>
<td>7.4</td>
<td>3.1</td>
<td>1.9</td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>35.6</td>
<td>14.4</td>
<td>9.1</td>
<td>11.6</td>
<td>5.1</td>
<td>3.1</td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>53.6</td>
<td>21.7</td>
<td>13.6</td>
<td>17.5</td>
<td>7.4</td>
<td>4.5</td>
<td></td>
</tr>
</tbody>
</table>

$w/b$ ratio

Table 4 - TCI values for MPT station

<table>
<thead>
<tr>
<th>$d$</th>
<th>$C_c$</th>
<th>$B_1$ bar</th>
<th>$B_2$ bar</th>
<th>$d$</th>
<th>$C_c$</th>
<th>$B_1$ bar</th>
<th>$B_2$ bar</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td>$w/b$ ratio</td>
<td>$w/b$ ratio</td>
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<td>$w/b$ ratio</td>
</tr>
<tr>
<td>25</td>
<td></td>
<td>0.4</td>
<td>0.5</td>
<td>0.6</td>
<td>0.4</td>
<td>0.5</td>
<td>0.6</td>
</tr>
<tr>
<td>30</td>
<td>18.2</td>
<td>7.1</td>
<td>4.5</td>
<td>7.0</td>
<td>3.0</td>
<td>1.8</td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>46.5</td>
<td>18.7</td>
<td>11.8</td>
<td>16.3</td>
<td>7.0</td>
<td>4.3</td>
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</tr>
<tr>
<td>40</td>
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<td>17.3</td>
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<td>37.4</td>
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<td>13.2</td>
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<td>64.4</td>
<td>27.2</td>
<td>17.0</td>
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<td>49.6</td>
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<tr>
<td>75</td>
<td>98.6</td>
<td>61.7</td>
<td>*</td>
<td>46.4</td>
<td>29.3</td>
<td></td>
<td>75</td>
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</table>

$w/b$ ratio
Large-size rebars proneness to higher chloride build-up lead to faster corrosion. The corrosion reaction sets in first at locations along the perimeters of rebars, where the chloride build-up is faster and higher. From tables 3 and 4 it can be observed that, for the given values of cover thickness and water-to-binder ratio, with the increase in the rebar size, TCI for B1 bar is faster by about 4\text{–}6\% between any two intermediate sizes (i.e. 12 and 16 mm, 16 and 20 mm, and 20 and 25 mm) and about 10\text{–}15\% between the extreme sizes (i.e. 12 and 25 mm).

Furthermore, from the numerical analysis it is also observed that, neither the corner bar nor the middle bar experiences corrosion in the other nine stations. This can again be attributed to the lower values of surface chloride content at these stations. For example, zero surface chloride content values are reported for two of the nine stations, namely, Mahendragiri and NIO; hence, rebars at these stations could hardly experience any corrosion.

The minimum cover thickness required to sustain a corrosion free service life of 50 years to the RC beam at Chennai and MPT stations for various parameters (rebar size and location, \(w/b\) ratio) are listed in Table 5 based on the estimated TCI values (Tables 3 and 4).

<table>
<thead>
<tr>
<th>Station</th>
<th>(w/b = 0.4)</th>
<th>(w/b = 0.5)</th>
<th>(w/b = 0.6)</th>
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<tbody>
<tr>
<td></td>
<td>35 – 40</td>
<td>55 – 60</td>
<td>55 – 60</td>
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<tr>
<td>MPT</td>
<td>25 – 30</td>
<td>35 – 40</td>
<td>35 – 40</td>
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</tbody>
</table>

It can be observed from Table 5 that the minimum cover thickness requirements are functions of both \(w/b\) ratio, and, rebar size and location. It can also be observed from Table 5 that, cover thickness requirements for Chennai station are higher than that for MPT station. As a part of the durability requirements, IS 456:2000 has classified coastal environment under severe exposure class and hence recommends a minimum cover thickness value of 40 mm and the maximum \(w/b\) ratio of 0.45 for RC components exposed to these environments. From the numerical analysis, it is observed that, IS 456:2000 cover thickness recommendations are sufficient to sustain a corrosion free service life of 50 years for the middle bars of the RC beam, however, are found to be inadequate for the corner bars. For example, minimum cover thickness for Chennai station ranges between 55–60 mm for the \(w/b\) ratio of 0.4, which are higher than the value recommended by IS 456:2000. Inadequate cover thickness can have serious implications on the durability requirements of RC components exposed to coastal environments. Moreover, insufficient cover thickness can result in premature deterioration resulting in early reconstruction or major repairs involving huge expenditures. In addition, premature deterioration may also lead to sudden failure of structures leading to loss of life and property (Ramalingam and Santhanam, 2012).
TIME-VARIANT CROSS-SECTIONAL AREA OF REBAR

To estimate the time-variant degradation of flexural capacity in the RC beam caused by rebar corrosion, the time-variant reduction in the net section of the rebar needs to be quantified. In the current study, the corrosion penetration depths around steel-concrete interfaces of rebars are estimated based on two of the corrosion rate models, namely, Vu and Stewart (2000) and BS 6349−1. The former is a time-variant corrosion rate model, whereas the latter is a constant corrosion rate model. Figs. 3(a) and (b) show the time-variant cross-sectional area of the longitudinal rebars of the RC beam, for various combinations of cover thickness (25 and 30 mm) and rebar diameter (12 and 16 mm), and having a $w/b$ ratio of 0.4.

![Fig. 3 - Time-variant cross-sectional area of the longitudinal rebars for $C_c = 25$ and 30mm:](image)

(a) $d = 12$ mm and (b) $d = 16$ mm

Similarly, Figs. 4(a) and (b) show the time-variant cross-sectional area of the longitudinal rebars of the RC beam, for various combinations of cover thickness (25, 30, and 35 mm) and rebar diameter (20 and 25 mm), and having a $w/b$ ratio of 0.5.

The following observations can be made from Figs. 3 and 4:

(i) In general, the estimated values of $A_s(t)$ based on Vu and Stewart (2000) model is much higher than that based on BS 6349−1. This can be attributed to relatively higher corrosion rate estimation by the former.

(ii) After the onset of corrosion reaction in the middle bars (B₂), the estimated values of $A_s(t)$ based on Vu and Stewart (2000) model has a diverging trend from BS 6349−1 model. This is because the former is built based on time-variant corrosion rate, whereas the latter is a constant corrosion rate model.

(iii) The corrosion reaction sets in first around the steel-concrete interfaces of corner bars than the middle bars. The values of $A_s(t)$ show a clear change in slope, when the middle bars (B₁) start corroding. This change in slope indicates increased reduction in the values of $A_s(t)$.

(iv) The effects of cover thickness on the values of $A_s(t)$ are more evident from the model of Vu and Stewart (2000), due to this model incorporating $C_c$ as one of the parameter.
TIME-VARIANT FLEXURAL CAPACITY

Eq. (9) relates the time-variant flexural capacity of the corroding RC beam with the time-variant cross-sectional area of the longitudinal rebars (Figs. 3 and 4). Figs. 5(a)–(c) show the time-variant flexural capacity of the RC beam, for various combinations of cover thickness (25 and 30 mm) and rebar diameter (16, 20 and 25 mm), and having a $w/b$ ratio of 0.4.

The following observations can be made from Fig. 5:

(i) In general, the estimated values of $M_{ur}(t)$ based on Vu and Stewart (2000) model is much higher than that based on BS 6349–1. This can be attributed to relatively higher estimations of $A_s(t)$ values by the former.

(ii) The differences between the values of $M_{ur}(t)$ estimated based on Vu and Stewart (2000) and BS 6349–1 models decreases with the increase in cover thickness. This is because the corrosion rate and hence $A_s(t)$ values estimated by the former decreases with increase in cover thickness.

(iii) Similar to the patterns of $A_s(t)$ observed in Figs. 8.5 and 8.6, the values of $M_{ur}(t)$ also show a clear change in slope, when the middle bars (B1) start corroding. This change in slope indicates increased reduction in the values of $M_{ur}(t)$. 
Similarly, Figs. 6(a)–(d) show the time-variant flexural capacity of the RC beam, for various combinations of cover thickness (35 and 40 mm) and rebar diameter (12, 16, 20 and 25 mm), and having a $w/b$ ratio of 0.5. The distributions of the values of $M_{\text{ult}}(t)$ illustrated in Figs 6(a)–(d) indicate almost identical behavior similar to the observations made in Fig. 5. In addition, as expected $M_{\text{ult}}(t)$ reduction is relatively higher, which can be attributed to the effects of corrosion accompanying the higher $w/b$ ratio (=0.5).

Finally, Figs. 7(a)–(d) show the time-variant flexural capacity of the RC beam, for various combinations of cover thickness (45 and 50 mm) and rebar diameter (12, 16, 20 and 25 mm), and having a $w/b$ ratio of 0.6. The distributions of the values of $M_{\text{ult}}(t)$ illustrated in Figs 7(a)–(d) indicate almost identical behavior similar to the observations made in Figs. 5 and 6. In addition, as expected $M_{\text{ult}}(t)$ reduction is relatively highest, which can be attributed to the effects of corrosion accompanying the highest $w/b$ ratio (=0.6).
SUMMARY AND CONCLUSIONS

In this study a numerical framework that can efficiently quantify time-variant degradation in the flexural capacity of a corroding RC beam based on non-uniform corrosion presented. This framework considered the effects of rebar size and location on the process of chloride ingress into concrete and focused on rebar corrosion due to atmospheric chlorides by considering diffusion as the dominant mode of chlorides ingress. Further, time to first onset of corrosion reaction for the corner and middle bars embedded in a RC beam are presented for 11 atmospheric exposure stations in India by incorporating various parameters such as $w/b$ ratio, concrete cover thickness and rebars size. Moreover, quantitative estimations of non-uniform reductions in the net cross-sectional areas and an assessment of the time-variant degradation in the flexural capacity of a corroding RC beam are evaluated based on non-uniform corrosion.

Fig. 6 - Time-variant flexural capacity of the RC beam with $w/b$ ratio of 0.5 and $C_c$ of 35 and 40 mm:
(a) $d=16$ mm, (b) $d=20$ mm and (c) $d=25$ mm
For the given values of cover thickness and water-to-binder ratio, TCI for B_2 bar is faster than that for B_1 bar. In addition, the ratio between TCI of B_1 and B_2 bar is observed to fall within the range of 1.3–1.6, indicating TCI for B_2 bar is 30–60% faster than that for B_1 bar. The larger the rebar, in general, the bigger the obstruction, and therefore, the higher the chloride build-up. The minimum cover thickness required to guarantee a certain number of corrosion-free service life is observed to increase not only with the increase in water-to-binder ratio but also with the rebar locations and sizes.

It is observed from the numerical analysis that the rebars in RC beam that are located in two of the 11 stations, namely, Chennai and MPT experience corrosion over the simulation period of 100 years. This is because, the surface chloride content values reported at these stations are relatively higher. In addition, it is also observed from the numerical analysis that neither the corner bar nor the middle bar experiences corrosion in the other nine stations. This can again be attributed to the lower values of surface chloride content at these stations. Moreover, estimated time-variant reductions in cross-sectional area of longitudinal rebar and flexural capacity of the RC beam show a clear change in slope, when the middle bars start corroding.
REFERENCES


