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## **Next Generation Reinforced Concrete Corrosion Modeling with Interdependent Initiation and Propagation Stages**

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### **ABSTRACT**

Most damage prediction models for reinforcing steel in concrete treat the corrosion propagation stage as an independent module, with little dependence on the prior history of corrosion development in nearby portions of the structure. A next generation interdependent model (IM) addresses those interactions for chloride-induced corrosion. The model incorporates the corrosion prevention effect of the local potential drop caused by prior active-to-passive transitions in nearby steel portions. Prior work demonstrated IM for 1D and 2D configurations but 3D realizations were needed. An exploratory 3D approach is presented here, toward incorporation in practical modeling forecasts. Confirming previous 1D and 2D models, the long-term damage projected for the 3D IM was only a fraction of that forecast using traditional potential-independent threshold modeling. A computational scheme using intermediate regular point grids was created to enable and speed up calculations. Issues requiring further development are identified.

Key words: Reinforced Concrete, Corrosion, Modeling, Interdependent initiation and propagation.

### **INTRODUCTION**

Corrosion of steel in reinforced concrete is a major durability limitation factor. Reinforcing steel bars (rebars) placed in concrete normally develop a passive surface condition due to the highly alkaline (e.g., pH >13) nature of the concrete pore water. Corrosion rates in that condition are extremely low ( $\ll 1$   $\mu\text{m/y}$ ) with no adverse consequences in regular practice. However, this situation often deteriorates on exposure to the service environment. In the most damaging form, addressed in the paper, chloride ions from marine or deicing salt penetrate through the concrete cover until the chloride concentration at the rebar surface exceeds a threshold value,  $C_T$ , that triggers passivity breakdown. As a result, corrosion rates at the

affected rebar surfaces greatly increase to values that can be well above 10um/y. Solid expansive corrosion products form then at or near the rebar-concrete interface, which cause increasingly larger local tensile stresses. Since concrete has low tensile strength (e.g., 1 MPa) and low fracture toughness, cracks form easily that soon can become apparent on the external concrete surface. The presence of the cracks, often associated with delamination or spalls of the concrete, degrades the structure. If the degradation is such that the structure can no longer provide its required service, for example as a smooth roadway surface, it is said that the structure or portion thereof has reached its serviceability limit state (SLS) and repairs or rehabilitation are needed.

Corrosion-related durability forecasts traditionally follow the approach attributed to Tuutti<sup>1</sup> whereby the service life  $t_s$  from placement into service until reaching the SLS is divided into a corrosion initiation and a corrosion propagation stage, of duration  $t_i$  and  $t_p$  respectively, so  $t_s = t_i + t_p$ . During corrosion initiation the rebar surface is in the passive condition but  $\text{Cl}^-$  concentration there increases as penetration from the outside progresses. The initiation stage ends when the  $\text{Cl}^-$  concentration reaches  $C_T$ . The propagation stage starts at that point, and continues while corrosion and associated cracking damage progresses, ending when the SLS is reached. To avoid overly optimistic evaluations, it is often conservatively declared that the SLS has been reached at a given location when the first crack appears there. Structures are extended sometimes over many km, so  $t_s$  forecasts tend to be divided into different parts the structure such as spans of a bridge, or structural components within a given part, or even small portions within a given component. Parameter variability is addressed by statistical treatments by various modeling approaches discussed in the following sections.

In its most elementary form, a single-value deterministic projection of  $t_s$  involves calculating  $t_i$  and  $t_p$  by assuming a mechanism with descriptive equations for each of the stages. The equations are then solved using as input the values of the parameters for each of the equations that are consistent with the structure and service conditions. Simplifying assumptions are used throughout. Thus frequently the penetration of chloride ions through the concrete cover is treated as if it were by Fickian diffusional transport in a homogeneous medium with a spatially independent diffusion coefficient  $D$  considered as a property of the concrete type and the exposure conditions. Similarly, simple boundary conditions such as a constant external chloride surface concentration  $C_s$  and zero initial concentration may be assumed, together with one-dimensional transport through a uniform concrete cover thickness  $x_c$ . For those particular assumptions, the duration of the initiation stage can be readily obtained from the solution to the diffusion equation for the conditions assumed whereby so  $t_i = f(C_s, D, C_T, x_c)$ . The values of  $C_s$ ,  $D$  and  $C_T$  may be known or estimated from prior measurements in the field or laboratory with similar environmental conditions and materials, and the value of  $x_c$  comes from direct measurement or design data. Likewise, the duration of the propagation stage may be estimated by assuming that external cracking manifestations take place when a given amount of corrosion product accumulation has taken place. Further assuming that the accumulation is proportional to the amount of radial corrosion penetration on the steel rebar, the SLS may be considered to have been reached when that penetration reaches a critical value  $X_{CR}$ . If the corrosion penetration under the exposure conditions is progressing at a corrosion rate  $CR$ , then the duration  $t_s$  is given by  $t_s = X_{CR}/CR$  if  $CR$  is constant, or by a solution of an equation integral on  $t_s$  if  $CR = f(t)$ . For a given concrete type and exposure regime the effective value of  $X_{CR}$  is also found to be dependent on rebar and corroding region size, pitting morphology and distribution on rebar surface, as well as on cover thickness,  $x_c$ , so more generally for a given exposure regime  $t_s = f(CR(t), X_{CR}(x_c, \text{other geometric factors}))$ . The values of the parameters for  $CR$  and  $X_{CR}$  may also be known or estimated, so combining  $t_p$  with the previously established initiation stage duration, a forecast for  $t_s = t_i + t_p$  can be completed. In an extended structure the value of  $t_s$  varies spatially and from element to element due to different micro-climates, and overall damage progression needs to take into account the statistical distribution of the parameter values that determine  $t_i$  and  $t_p$  in each portion of the structure.

Traditional damage prediction models for reinforcing steel in concrete treat the corrosion propagation stage as a module independent of that determining  $t_i$ , with little dependence on the prior history of

corrosion development in nearby portions of the structure. As noted in previous work <sup>2</sup> such approach can be severely limited, because it ignores electrochemical coupling that can seriously influence damage progression. Here a next generation interdependent model (IM) for corrosion forecasting is advanced that considers the electrochemical interactions disregarded in traditional models. The model incorporates the corrosion prevention effect of the local potential drop caused by prior active-to-passive transitions in nearby steel portions, as well as traditional protection caused by cathodic coupling with previously corroding nearby elements. The term cathodic prevention alludes to the increase in  $C_T$  that is experienced by steel when its potential becomes more negative while still in the passive condition. The phenomenon is in part related to the well know inverse trend, for passive alloys, between pitting potential and  $[Cl^-]$  in aqueous solutions. Thus for a given  $[Cl^-]$  the alloy stays passive as long as its potential with respect to the solution is below the pitting potential, and the  $[Cl^-]$  needed for pitting initiation increases the more negative the polarizing potential. Depending on transport conditions, the effect can be enhanced by local increase in pH from production of  $OH^-$  ions upon cathodic polarization. In corrosion of rebar in concrete, activation of the steel is usually accompanied by a substantial drop in potential at the newly created anode. Electrolytic coupling with the surrounding passive steel causes its potential to become somewhat more negative than before, so the local value of  $C_T$  on the surrounding passive steel increases accordingly as noted above. Thus as chloride contamination increases, subsequent corrosion activation events in the surrounding steel are delayed. Corrosion rate at the initial anode region can be aggravated however due to an adverse cathode-anode area ratio regime.

Those phenomena tend to result in long-term IM damage projections that are significantly lower than for conventional modeling, indicating that the latter can be excessively conservative. In contrast, there is also the local aggravation of corrosion in portions where corrosion had first initiated, which traditional models tend to neglect as well. Given the importance of these outcomes it is highly desirable to implement IM in durability modeling. However, IM is computationally challenging, requiring simultaneous iterative updates of corrosion computations to determine potential distribution, and chloride ingress. IM has been earlier demonstrated for less realistic but computationally more tractable 1D and 2D configurations approximating marine substructure<sup>2-4</sup>. The results confirmed the expected radical departures from output of conventional non-interactive models, encouraging expanding the approach to more realistic 3D systems. Here a simplified realization of a reinforced concrete slab is presented for a chloride-induced corrosion scenario, as an initial step toward incorporating this IM approach in practical 3D modeling forecasts. The objective of this work is to uncover the challenges and develop initial strategies to achieve a tractable implementation of 3D IM.

## MODELED SYSTEM AND ASSUMPTIONS

The model simulates corrosion damage progression in a reinforced concrete slab of uniform width, depth and height  $X, Y$  and  $Z$  respectively along coordinates  $x, y$  and  $z$ . The steel reinforcement is located near the bottom of the slab. The concrete is initially chloride-free but the top face of the slab is exposed to chloride salt from the service environment. The slab sides are idealized as being physically isolated, effectively treating it as a unit element of a larger slab with uniform exposure to avoid arbitrary edge effects. The following simplifying idealizations are used to make the problem tractable and solvable in a practical computation time with an ordinary desktop computing system. However, these simplifications are not conceptually limiting and are amenable to greater complexity with adequate computing power and software optimization.

1. The concrete is approximated as a homogeneous medium with time and space independent electric conductivity  $\sigma$  and diffusion coefficient of chloride ions  $D$ . Thus within the concrete domain the electric charge and chloride mass conservation equations are

$$\nabla^2 V = 0 \quad (1)$$

$$D \nabla^2 C = \partial C / \partial t \quad (2)$$

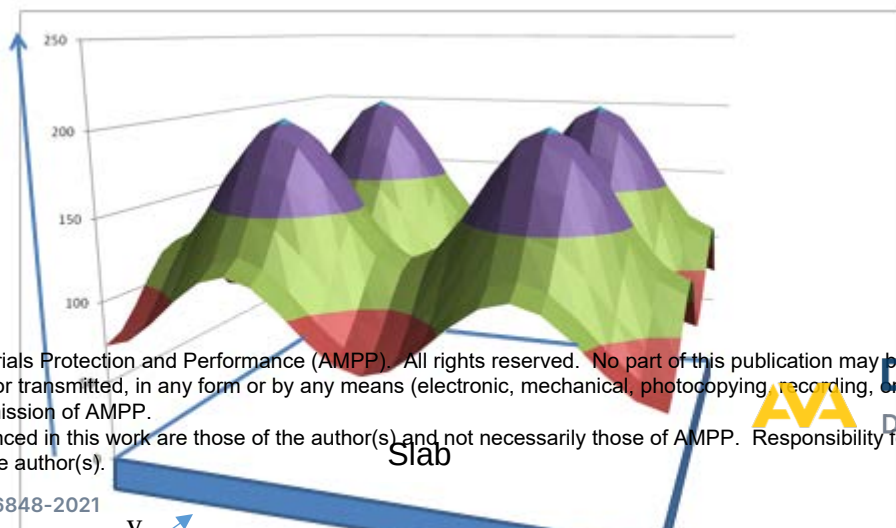
where  $V^A$  and  $C$  are the electric potential in the concrete and the homogenized chloride ion concentration in the concrete respectively. Ionic transport is affected by the presence of electric fields in the concrete so equations (1) and (2) are only working approximations ignoring such interactions, in lieu of more detailed future treatments.<sup>5</sup>

Likewise the electric current density vector  $J$  is given by

$$J = -\sigma \nabla V \quad (3)$$

2. To enable the next simplification step, the steel rebar mesh location is considered to be fully to the bottom of the slab. This placement disregards the small electric coupling or chloride storage capacity of the relatively thin concrete region that exists between the rebar mesh and the bottom of the slab. The bulk of the chloride transport and electric coupling effects are still captured by the overall simulation.
3. The rebar mesh assembly is assumed to have a placement density such that the total amount of steel surface area is comparable to the size of the slab footprint, as is often encountered in actual practice. Thus, for the purposes of simulating the extent and effects of corrosion surface phenomena, the assembly is further abstracted as equivalent to having a steel sheet at the bottom of the slab, with corrosion taking place on the side of the sheet in contact with the concrete. Similar approximations have been successfully used in previous modeling of reinforced concrete corrosion.<sup>6</sup>
4. Chloride ion concentration on the upper surface of the concrete slab is approximated as being a time-invariant amount  $C_s(x,y)$ . The environmental chloride exposure normally causes a gradual increase of chloride concentration at the surface of the slab until a steady state is reached, albeit with seasonal variations. Here for simplicity, it is assumed that the steady state has been reached early on in the life of the structure and that seasonal variations are small. Similar assumptions have been used successfully in simulations of marine exposure regimes.<sup>7</sup>
5. While time-independent,  $C_s$  is assumed to be position-dependent, with some parts of the surface being richer in  $Cl^-$  content than others. That situation is often encountered for example on a bridge deck where some parts of the roadway experience greater brine accumulation, or in a marine wall with uneven seawater splash. Under those circumstances, and others like uneven concrete cover or permeability, corrosion initiation events become spread in time and space, which is rather the norm in real world structures.<sup>8</sup> Here the variability was represented (Figure 1) by choosing a doubly sinusoidal distribution of  $C_s$  with pronounced maxima among intermediate minima of about half the value of the maxima. This distribution, which also serves as a proxy for other sources of corrosion initiation variability, was chosen for convenience to highlight the model features. The choice is not limiting however as input can include any  $C_s$  distribution either in analytical or numerical form.<sup>2, 4-5,9</sup>

$C_s$  (mol/m<sup>3</sup>)



<sup>A</sup> To be consistent with the model, the concrete is understood as being connected to the steel rebar terminal is connected to (SCE)), with the tip of the point is inside the concrete manner, where the voltage

point in the terminal is the plus electrode as well if the chemical