ABSTRACT

CALDERON, VICTOR ALEJANDRO. Time Dependent Performance Based Design. (Under the direction of Dr. Mervyn Kowalsky.)

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Time Dependent Performance Based Design

by
Victor Alejandro Calderon

A research proposal submitted to the Graduate Faculty of
North Carolina State University
in partial fulfillment of the
requirements for the Degree of
Doctor of Philosophy

Civil Construction and Environmental Engineering

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Chapter 1

INTRODUCTION

Bridges are designed based on discrete events with minimal consideration of interactions between hazards/loading, material aging (or more accurately condition) and bridge performance. The purpose of the research described is to study Time Dependent Performance Based Design that considers the effects of cumulative damage on the properties of the materials both as a function of time and current condition. Specific items of interest include corrosion, strain aging, low cycle fatigue and strength aging. In addition, since there is a high likelihood for a structure in a high seismic region to be subjected to more than one main shock throughout its life, it is deemed important to consider the effects of multiple earthquakes. As a consequence, the effects of repairs on the structural response are also of great importance. An analytical procedure is implemented such that it considers the effect of aging on structures, more specifically this study starts by evaluating an RC bridge Column. A series of condition dependent nonlinear time history analysis are performed assuming that a series of earthquakes occurs throughout the lifetime of the structure while at the same time changing the properties of the structure as time progresses. To achieve this a library of time dependent materials are developed. At the end of each series the main variables of study are the the limit state that was reached, the controlling mode of response (flexural or shear controlled), Equivalent Viscous Damping and the accumulated deformations. The series of earthquake proposed consists of (1) equally spaced main shocks only, (2) main shock-aftershocks series and (3) main shock-aftershock-repair series. At the end of the presentation recommendations on design of new structures and assessment of existing structures will be provided.

1.1 Motivation

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Chapter 2

LITERATURE REVIEW

In this chapter the available knowledge on the different topics that are available in the literature are summarized. First a review on the different definitions of commutative damage is presented then the main idea for this research are established and the required components, then the different elements that form part of this study are presented and then a general concept is established and presented in Chapter 3.

2.1 Cumulative Damage

Cumulative Damage in structures have been tried to be established for structures to identify the state of a structure

The best-known and most widely used of all the cumulative damage index is that of Park and Ang (1985). This consists of a simple linear combination of normalized deformation and energy absorption:

The first term here is a simple, pseudo-static displacement measure. It takes no account of cumulative damage, which is accounted for solely by the energy term. The advantages of this model are its simplicity, and the fact that it has been calibrated against a significant amount of observed seismic damage, included some instances of shear and bond failures. Park, Ang and Wen (1985) suggested D = 0.4 as a threshold value between repairable and irreparable damage, while the same authors in 1987 suggested the following more detailed classification:

2.1.1 Damage Index

2.1.2 Fragility Curves

Chapter 3

Study Gap

3.1 Research Gap

3.2 Objectives

3.2.0.0.1 Filler Text

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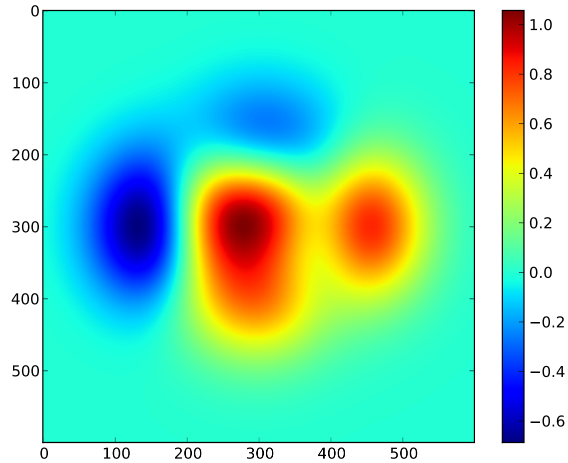

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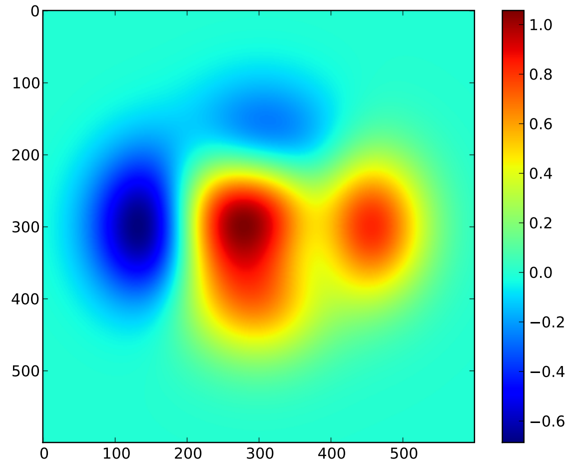

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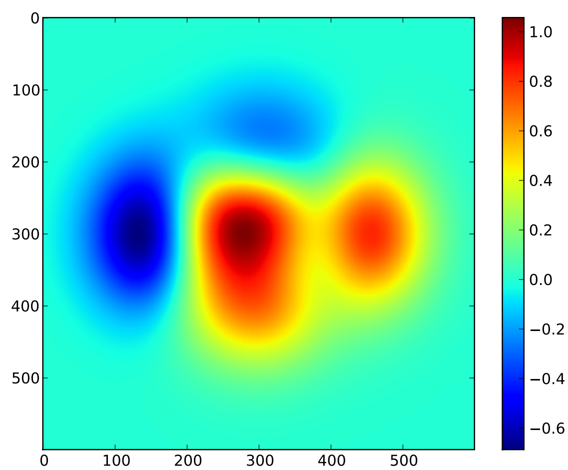

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Chapter 4

Methodology

A methodology that incorporates the different sources of cumulative damage in RC structures is proposed, the main elements that might induce damage in a structure are:

- Corrosion
- Strain aging
- Low-cycle fatigue
- Concrete Strength Aging

This effects generally affect the mechanical behavior of the material, which are nor considered when designing a structure. In the following paragraph the different models available are studied and later incorporated into the analysis methodology.

4.1 Corrosion

One of the main phenomenon that affect the long term behavior of RC structures is corrosion of the reinforcing steel. Two types of corrosion are possible:

- Carbonation,
- Chloride attack

The main source of corrosion in most RC structures is Chloride Attack and is the one that is assumed in the present study.

Corrosion of steel in concrete is an electrochemical process [5] this corrosion mat be generated in two ways:

- Composition cells may be formed when to dissimilar metals are embedded in concrete or when significant variations exist in the surface characteristics of steel
- In the vicinity of steel concentration cells may be formed due to differences in the concentration of dissolved ions, such as alkalies and **chlorides**.

The corrosion process under chloride attack type of corrosion consists in first the protective film on the reinforcing steel surface is destroyed, a process known as **depassivation**, then initiation of corrosion happens, the electrical resistivity and the oxygen content control corrosion. Figure 4.1 schematically show this process.

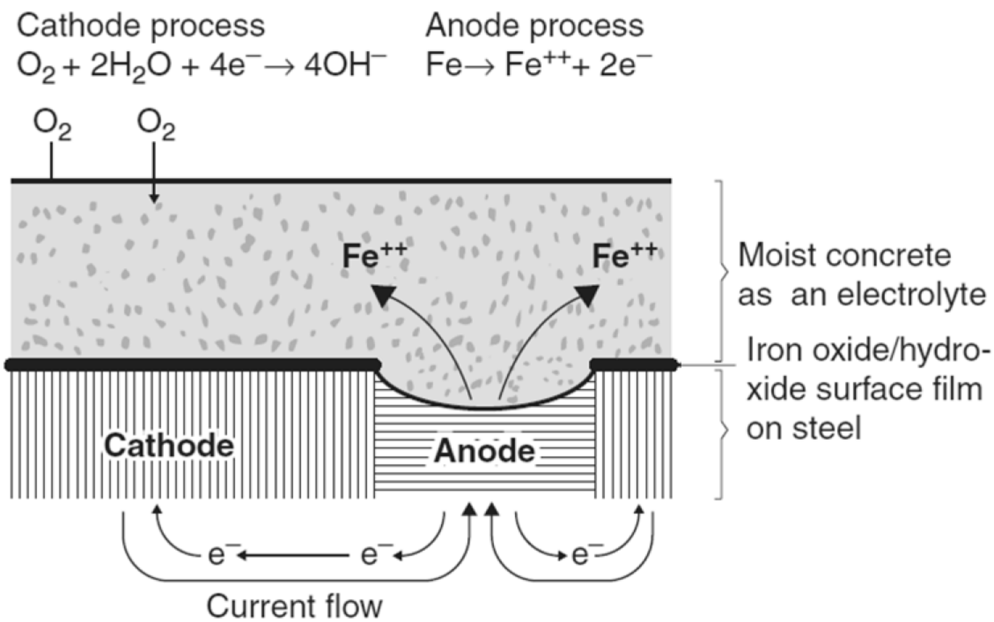

Figure 4.1 Corrosion Process in Reinforcing Steel Bar [5]

A literature review to characterize corrosion in reinforcing steel is presented such that corrosion can be modeled as a function of time, the corrosion process is an extensive field of research and to characterize it, therefore the different components of this subject are categorized as follows:

1. Time to Initiation of Corrosion (Tcorr)

2. Corrosion growth in reinforcing steel
3. Mechanical Properties of Corroded Reinforcing Steel (fycorr, fucorr)
4. Cyclic Test on Corroded RC Columns
5. Flowchart of Corrosion Model Implemented

4.1.1 Time to Corrosion

Time to corrosion refers to the corrosion initiation at which the passivation of steel is destroyed and reinforcement starts corroding actively.

Christensen Model

Christensen [8] main goal was to generate a corrosion model that was general for all concrete elements, additionally the authors tried to generate a model that also included the appearance of cracks due to corrosion that would evetually grow and the spall the concrete. More specifically related to reinforcing steel corrosion they developed a model based on Fick's law of diffusion to model the rate of chloride penetration into concrete as a function of concrete cover and time.

$$\frac{\partial C(x,t)}{\partial t} = D_c \frac{\partial C(x,t)}{\partial x^2} \tag{4.1}$$

After solving equation 4.1 the following expression results:

$$T_{corr} = \frac{d^2}{4D_c}\left[erf^{-1}\left(\frac{C_{cr} - C_0}{C_1 - C_0}\right)\right] \tag{4.2}$$

d: Concrete cover
D_0: Diffusion coefficient
C_0: Equilibrium Chloride Concentration
C_{cr}: Critical chloride corrosion concentration

While this model provides a means to calculate the Time for initiation of corrosion as a function of Concrete Cover and Diffusion concentration, the estimation of the Diffusion concentration depends on several factors such as environment, curing and water to cement ratio it is not a reliable method to estimate the Time to Corrosion.

Gosh & Padgett Model

Gosh et al calculate time to corrosion based on Thoft-Christensen model, considering in-field corrosion related studies of existing bridge components in the United States exposed to deicing salts to obtain mean values of chlorides concentration and put them in a modified version of the Thoft-Christensen Model.

$$T_{corr} = \frac{x^2}{4D_c}\left[erf^{-1}\left(\frac{C_0 - C_c r}{C_0}\right)\right]^{-2} \tag{4.3}$$

D_c : Diffusion Coefficient a recommended value of $1.29\frac{cm^2}{year}$ is given in their study [2]
C_0 : Surface Chloride Concentration, recommended (0.10)
C_r : Critical Chloride Concentration, recommended (0.04)

While this model provides mean values for the time of initiation of corrosion, it is limited to environments that are controlled by **dicing salts only**.

Life 365

Is a software developed by a consortium of companies of the cementitious materials industries and academic institutions. This software relies on the studies summarized above, mainly using the Thoft-Christensen model, but as opposed to assuming dicing environments only, this software uses a database of chlorides concentration for different location in the USA and Canada, which gives more accurate results depending on the location and environment in which the structure is located..

While this is a more robust model to obtain the initiation of corrosion since it considers the location and environment of the structure and it also has the ability to include other durability issues, it is difficult to implement in a batch run format since the program is closed.

Liu & Weyers Model

This model tries to calculate the time to initiation of corrosion by calculating the amount of corroding products that are needed to fill the voids in the concrete cover that will eventually generate cracking in this area and therefore initiate the accelerated corrosion of the reinforcing steel this is characterized through the following set of equations:

$$W_{crit} = \rho_{rust}\left[\pi\left[\frac{Cf_t'}{E_{ef}}\left(\frac{a^2+b^2}{a^2-b^2}+\nu_c\right)+d_o\right]D+\frac{W_{st}}{\rho_{st}}\right] \tag{4.4}$$

$$T_{cr} = \frac{W_{crit}^2}{2k_p} \tag{4.5}$$

$$k_p = 0.098(\frac{1}{\alpha})\pi D i_{corr} \tag{4.6}$$

W_{crit}: Critical amount of corrosion needed to induce cracking.
W_{st}: Mass of corroded steel.
ρ_{rust}: Density of rust material.
ρ_{st}: Density of steel.
f_t': Tensile strength of the concrete.
E_{ef}: Effective elastic modulus of concrete $E_{ef} = \frac{E_c}{1+\phi_{crit}}$
ϕ_{crit} Creep coefficient of the concrete.
D: Diameter of bar.
d_o: Thickness of pore band around the steel/concrete interface.
ν_c: Poisson's ratio of concrete.
C: Cover depth
$a = \frac{D+2d_o}{2}$
$b = C + \frac{D+2d_o}{2}$

This model however is limited to corrosion on concrete slabs, since a series of experiments were developed to generate these equations. This model howver was able to predict the time to corrosion with great accuracy.

4.1.2 Rate of corrosion

Vu et al. Model

To estimate the loss of steel cross section due to corrosion a time dependent corrosion rate model was developed by [9], this model implies that corrosion diminishes with time. As corrosion accumulates with time around the steel, it precludes the uncorroded steel to react with the environment. The model is shown in Eq. 4.7.

$$i_{corr} = \frac{37.5(1-w/c)}{d_c} \tag{4.7}$$

w/c: Water Cement ratio d_c: Cover depth

In Fig. 4.2 the behavior of this model for different values of w/c ratios is shown. It can be seen that at larger values of cover depth the rate of corrosion decreases rapidly and as the water cement ratio increases the rate of corrosion decreases.

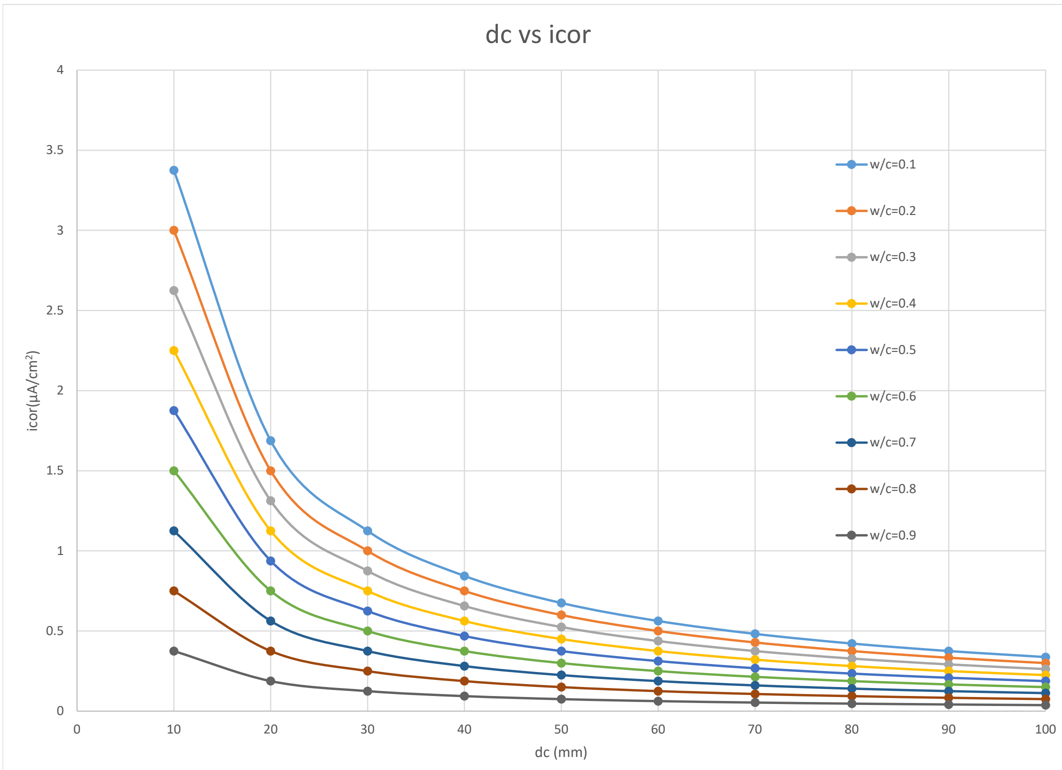

Figure 4.2 Concrete cover depth vs rate of corrosion

From the Vu et al model the diameter degradation is calculated according to Choe et al as:

$$d_{corr} = d_{bi} - \frac{1.0508(1 - w/c)}{d_c}(t - t_{corr})^{0.71} \tag{4.8}$$

d_{bi}: Is the initial diameter of the bar

The diameter is plotted in Fig. 4.3.

These values would correspond to a level of corrosion that varies from 7% corrosion to 21% of corrosion for w/c ratios that ranges from 0.4 to 0.6 The level of corrosion is calculated as:

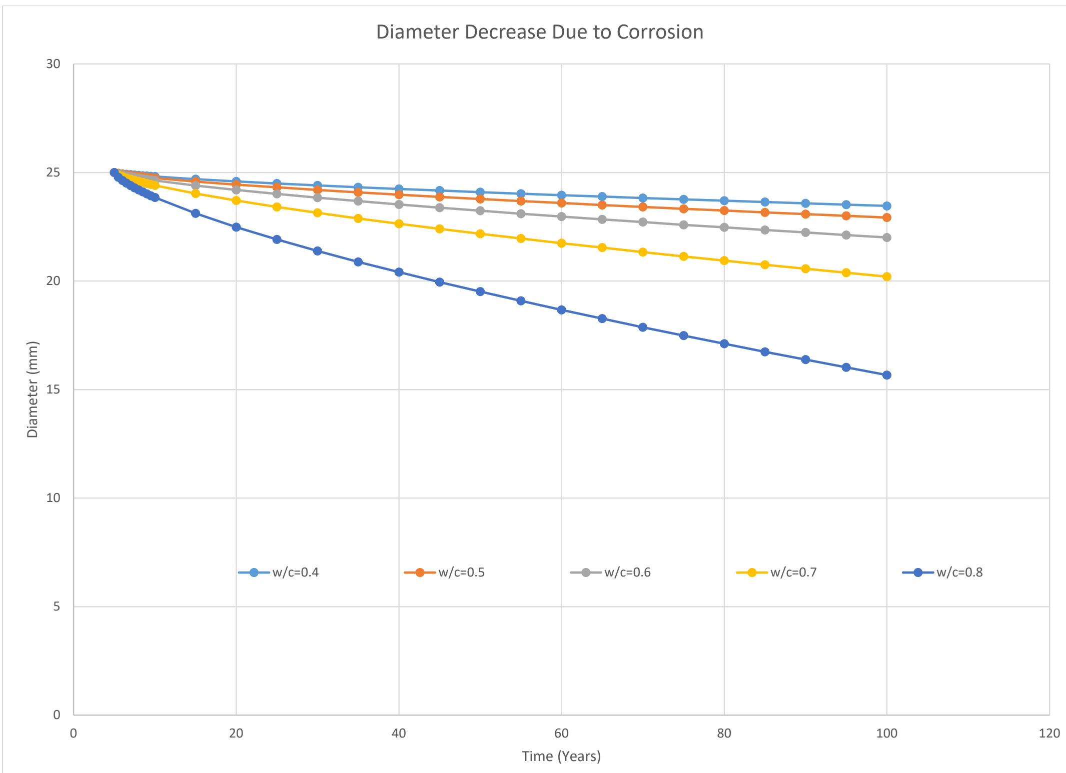

Figure 4.3 Diameter decrease due to corrosion

$$C = \frac{G_o - G}{g_o l_o} * 100 \tag{4.9}$$

Then the Corrosion level is plotted as a function of time in Fig. 4.4

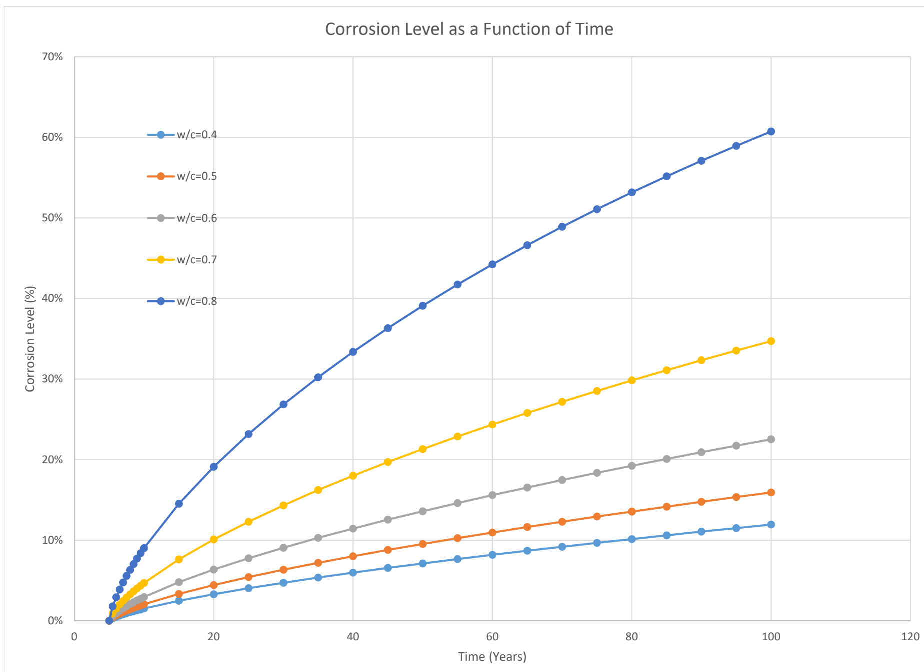

Figure 4.4 Corrosion Level vs Time (years)

4.1.3 Corrosion modified properties of reinforcing steel bars

In a study presented by Yuan et al [11] it was shown from experimental results that the mechanical properties of steel for different levels of corrosion could be modified for analysis as follows:

$$f_{y,C} = f_{yo}(1 - 0.021C) \tag{4.10}$$

$$f_{u,C} = f_{yo}(1.018 - 0.019C)$$

$$\delta_{s,C} = \delta_{so}(1 - 0.021C)$$

$$\varepsilon_{y,C} = \varepsilon_{yo}(1 - 0.021C)$$

Choe et al. Model Choe et al research is a seismic fragility estimates for RC columns subjected to corrosion, while the study is probabilistic in nature it defines the reduction in rebar cross section as:

$$d_b(t) = d_{bi} - 2\int_{T_{corr}}^{t} \lambda(t)dt \tag{4.11}$$

Considering the model proposed by Vu et al the bar diameter degradation can be expressed as:

$$d_b(t) = d_{bi} - \frac{1.508(1 - \frac{w}{c})^{-}1.64}{d}(t - T_{corr})^{0}.71 \tag{4.12}$$

Where the diameter of the bar and the cover is in (mm). Pros: Easy way to calculate the reduction of bar diameter. Cons: The model carries out the assumptions made by Vu et al. concerning concentration of chlorides assumed and the diffusion assumed.

With this information, the corrosion level is calculated as:

$$CL = \frac{d_i - d(t)}{d_i} \tag{4.13}$$

4.1.4 Physical test on corroded RC Structures

Recent studies [3], [4] and [10] have been developed to assess the force-displacement relationships in cantilever RC Columns. These columns were subjected to Quasi-Static Loading Protocol, the concrete columns were subjected to accelerated corrosion to obtain different Corrosion Levels (CL), the range of CL for these studies correspond to CL = 0%20%. In these studies the accelerated corrosion was performed via an electrochemical process directly applied to the reinforcing steel as shown in Fig. 4.5.

The resulting force displacement of these experiments is shown in Fig. 4.6 it can be seen that there is a reduction not only on the strength of the system but also on the displacement capacity.

As stated in the previous section the mechanical properties of steel are affected by corrosion,

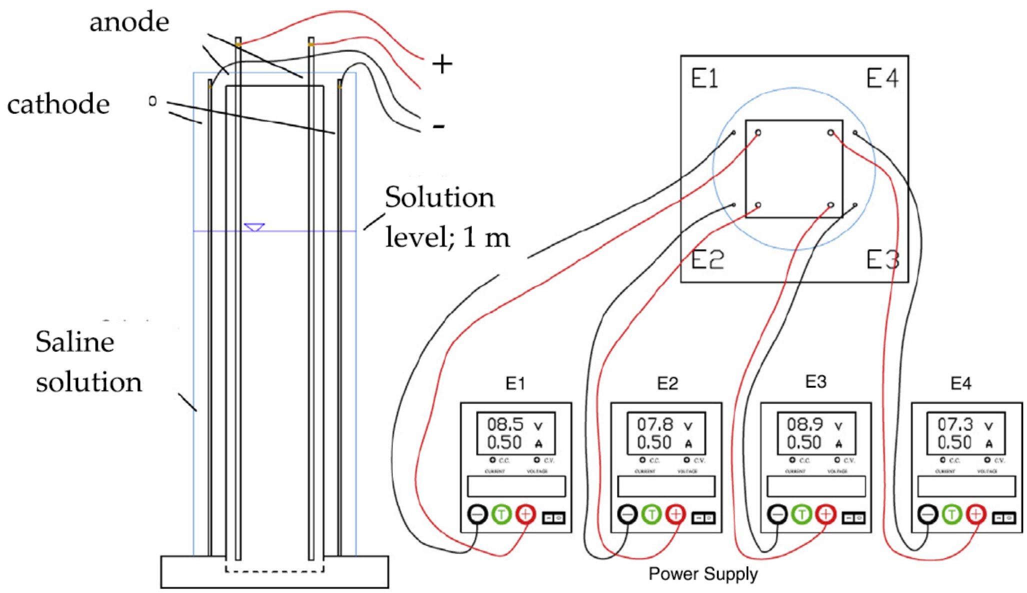

Figure 4.5 Corrosion Process for RC Column [4]

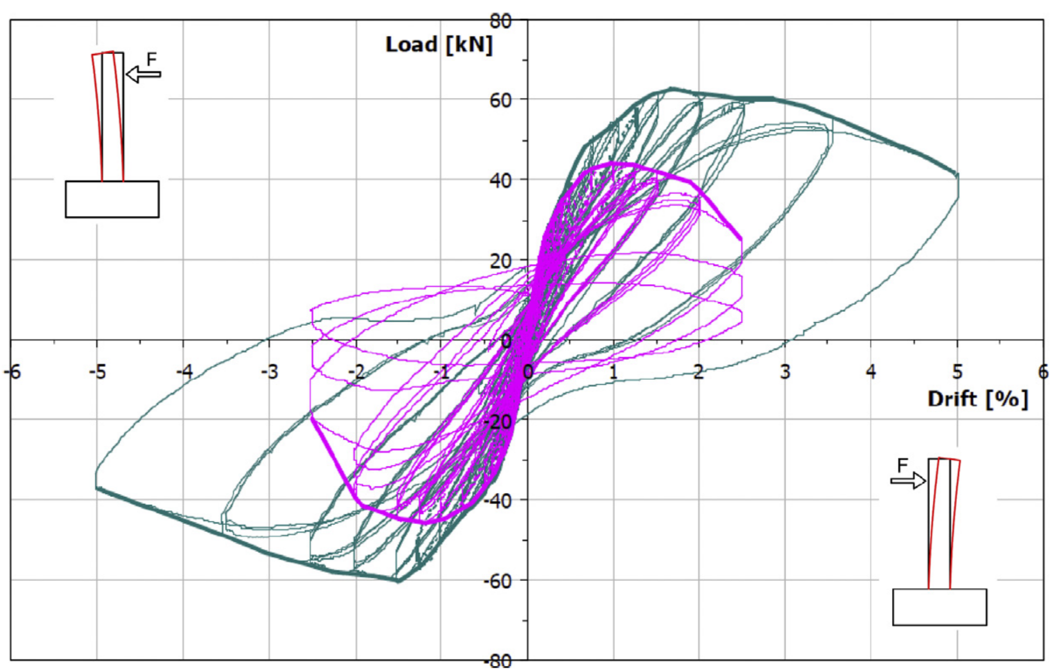

Figure 4.6 Corrosion Process for RC Column [4]

in the previous studies [4] the authors performed tension tests on corroded reinforcing steel. In these tests a reduction in the mechanical properties of steel was observed as well as a reduction in the rupture strain ε_{srup}, see Fig. 4.7.

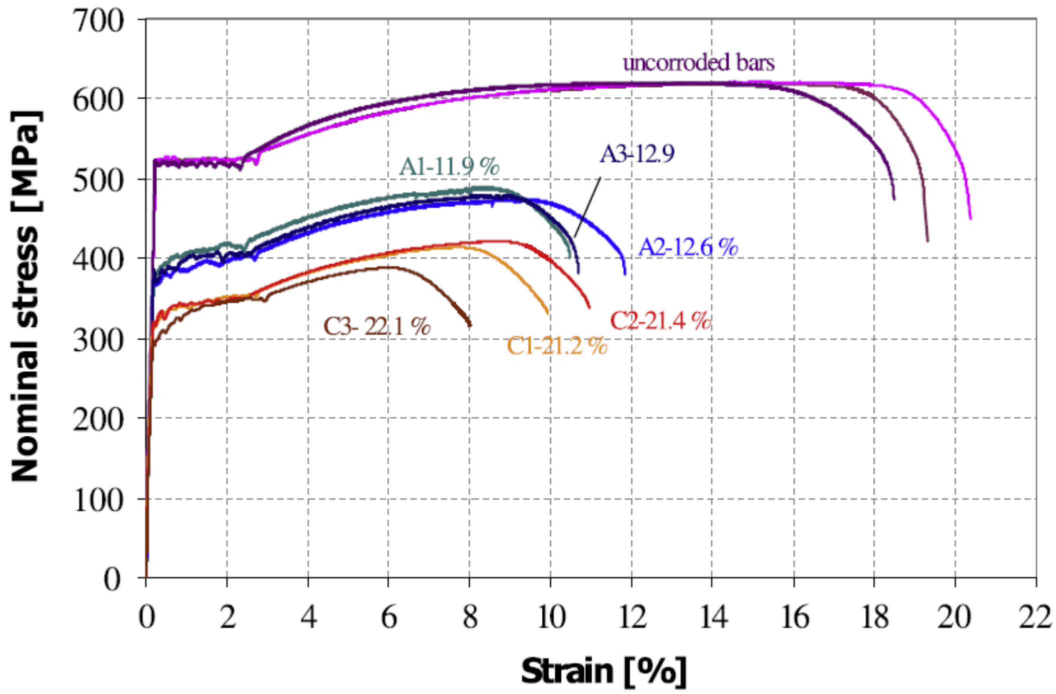

Figure 4.7 Corroded Rebars Stress-Strain Curves [4]

These studies did not considered the generation of the protective film due to the alkaline environment of the concrete, this film can modify mechanical properties of corroded steel. Additionally the accelerated corrosion process used a 3% $NaCl$ concentration solution while the chloride attack in concrete usually has a 1.0% - 1.5% concentration of the same Chloride. Therefore the results obtained from these studies might not accurately represent the actual conditions of corroded RC columns, thus an experimental campaign is proposed that would shed a light into the properties of corroded reinforcing steel inside concrete and is discussed in the following subsection.

4.1.5 Proposed Experimental campaign

As explained in the previous section the steel inside concrete generates a protective film and after chloride attack reaches the surface of the steel, this protective film starts to be eliminated. This same process will be simulated through the following steps.

1. Passivation of reinforcing steel

2. Accelerated corrosion of Reinforcing Steel

3. Tension Tests

4. Buckled Bar Tension (BBT) Test

Passivation of reinforcing steel

Methods to generate the passive film on reinforcing steel are available in the literature [1]. According to this study it is possible to generate the passivation process in the same way as it occurs to reinforcing steel inside the concrete. A porous solution will be generated with the following concentrations:

- $Ca(OH)_2$ Saturated

- $Na(OH)$ 4.00 g/l

- $K(OH)$ 11.22 g/l

- $Ca(SO)_4 + 2H_2O$ 13.77 g/l

The rebars will be placed in a container with the pore solution for a minimum of 8 days. Annodic Polarization Tests will be measured on the rebars to determine the passive current density. A figure of this process is shown in Fig. 4.8. Additionally The ends of the rebars will be protected to prevent corrosion in these zones of the specimens, the protection at the ends is based from the standard ASTM G109-07 with some alterations. Figures Fig. 4.9 and Fig. 4.10 show the specimen geometry and the preparation of the ends of the rebars.

Accelerated corrosion of Reinforcing Steel

The accelerated corrosion will be done by using a galvanic cell. Different studies [1] has shown that for rebars with pasive films a concentration of 0.3 Moles of sodium chloride ($NaCl$) will start the depassivation process on the rebars. The rebars will be subjected to a current of . This current is sustained for a period of time according to Faraday's Law until the desired level of corrosion is reached:

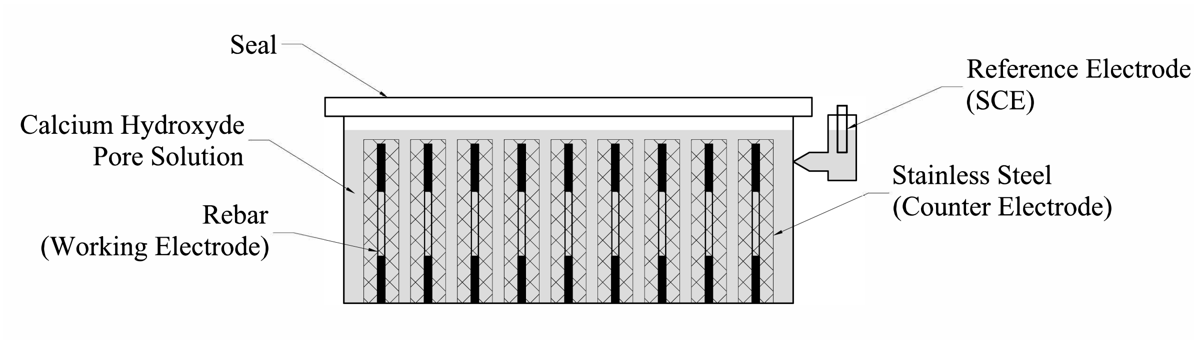

Figure 4.8 Rebars Passivation Process in Calcium Hydroxyde Pore Solution

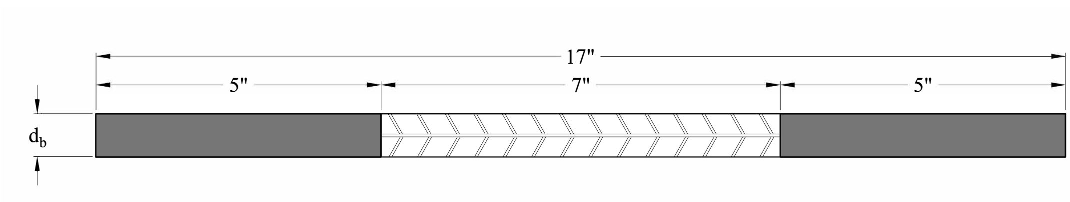

Figure 4.9 Rebar Specimen Geometry

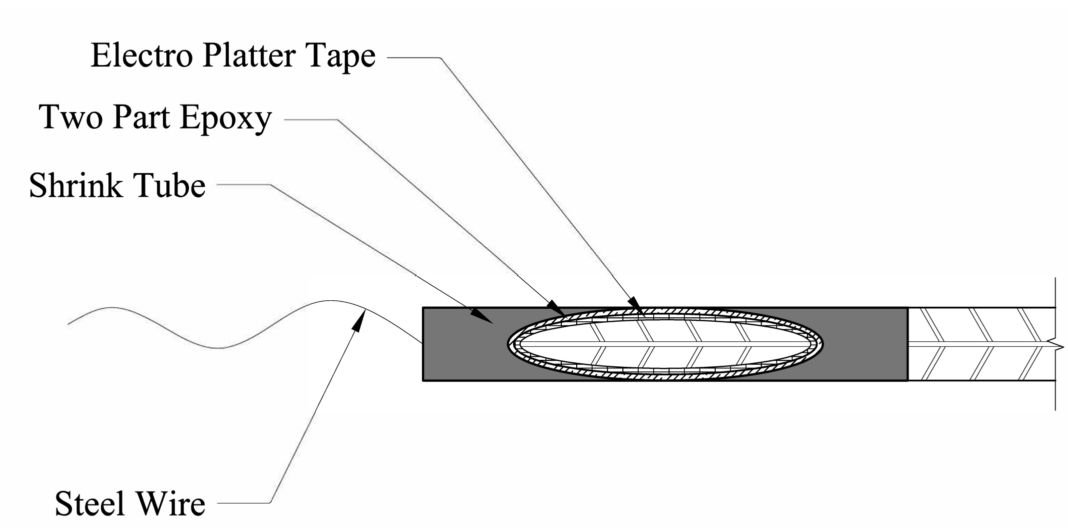

Figure 4.10 Rebars Ends Protection

$$t = \frac{\lambda m_{loss} \eta_{specimen} C_{faraday}}{i M_{specimen}} \tag{4.14}$$

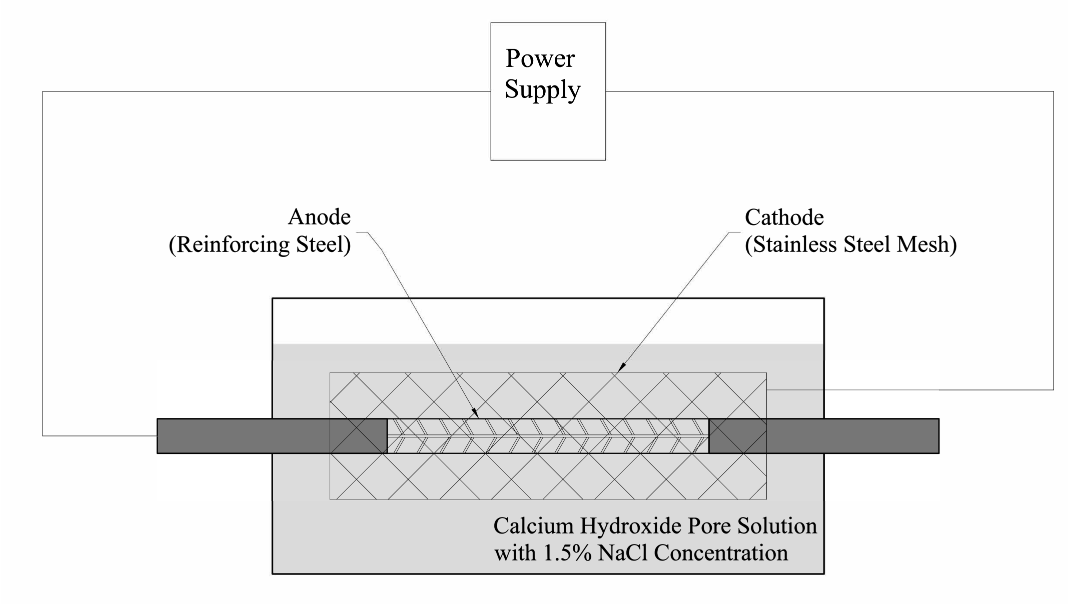

Figure 4.11 Accelerated Corrosion Process

For the different rebar sizes and Corrosion levels the current and the Time of Application is shown in Table 4.1. A current of $15mA$ is applied to the specimen obtaining the time this current needs to be applied for.

Table 4.1 Accelerated Corrosion to achieve Corrosion Levels.

Corrosion Level (CL)	Mass loss (g)	time(days)
5%	1.12	3
10%	2.24	6
15%	3.36	9
20%	4.47	12
25%	5.59	15

Tension Tests

Tension tests will be performed according to ASTM A715. The main objective of this tests is to evaluate differences in the Stress-Strain behavior of corroded Reinforcing Steel. This will help in determining any reduction in the ductility of steel for this condition.

Buckled Bar Tension (BBT) Test

One of the limit states that control Performance Based Design is Buckling of Reinforcing steel, recent tests have been developed to determine the behavior of buckling of reinforced steel (cite Leo and Robyn here). However those tests and those limit states have been developed for rebars in pristine condition, it is therefore necessary to check if available expressions to determine this limit state hold for corroded steel.

The Buckled Bar Tension Test consists in:

Test Matrix

4.2 Steel Strain Aging

4.2.1 Metallurgical Process

It is generally accepted that strain aging is due to the diffusion of carbon and/or nitrogen atoms in solution to dislocations that have been generated by plastic deformation. Initially, an atmosphere of carbon and nitrogen atoms is formed along the length of a dislocation, immobilizing it. Extended aging, however, results in sufficient carbon and nitrogen atoms for precipitates to form along the length of the dislocation.

These precipitates impede the motion of subsequent dislocations, and result in some hardening and loss in ductility. The extent of strain aging, which is a thermally activated process, depends primarily on aging time and temperature. In general, extended aging results in a saturation value above which further aging has no effect.

A second strengthening mechanism occurs when cold deformation (alone) is applied to steels. When dislocations break away for their pinning interstitial atoms and begin the movement causing slip they begin to intersect with each other. A complex series of interactions between the dislocations occurs, causing them to pin each other, decreasing their mobility. The decreased mobility also results in higher strength, lower ductility and lower toughness. As a result, cold deformed steels already have lowered ductility and toughness before any strain aging occurs and when heating follows cold deformation, the loss in ductility and toughness is greater. It is this combination of events that is the most damaging to the toughness of structural steels.

4.2.2 Strain aging effects in structures

Since it has already been established that strain aging is the process in which steel after being subjected to large strains develops an increased strength and reduced ductility with time and therefore important to include it in a time dependent analysis, considering the fact that plastic hinges will form in a ductile structure and the steel could reach high strains in this regions of the structure. Furthermore strain aging will cause an increased in the strength of the plastic hinge and as a consequence plastic hinges might be formed in regions of the structures that have not been designed for such demands. The effects of strain aging may also alter the transverse reinforcement due to both cold bending, making them susceptible to brittle failure.

According to [7] most strain aging occurs in the first 37 days. Also [6] studied strain aging effects with respect to time for different levels of pre-strains that ranged from $2\varepsilon_y - 10\varepsilon_y$ and for a time frame of 3 days to 50 days, from this study it was determined that a significant effect of strain aging took place from pre-strains $5\varepsilon_y$ and on. Strains higher than $15\varepsilon_y$ indicate a performance level in which substantial damage has been induced in the structure such that it is deemed unrepairable and therefore pre-strains higher that $15\varepsilon_y$ are unpractical and not studied by Montahan et al[6].

Momtahan et al Strain Aging Effects in Yield Strength of Steel

Momtahan et al was able to correlate the increase in yield strength as a function of time and the pre-strain in reinforcing steel bars. The proposed equations are shown below:

For $10\varepsilon_y$

$$\frac{f_y}{f_{yi}} = 0.0026t + 0.9838 \tag{4.15}$$

For $5\varepsilon_y$

$$\frac{f_y}{f_{yi}} = 0.0008t + 0.996 \tag{4.16}$$

For $2\varepsilon_y$

$$\frac{f_y}{f_{yi}} = 0.0004t + 0.9979 \tag{4.17}$$

It is proposed to limit the increase in yield strength to the one obtained at 50 days. These equations are plotted in Fig. 4.12

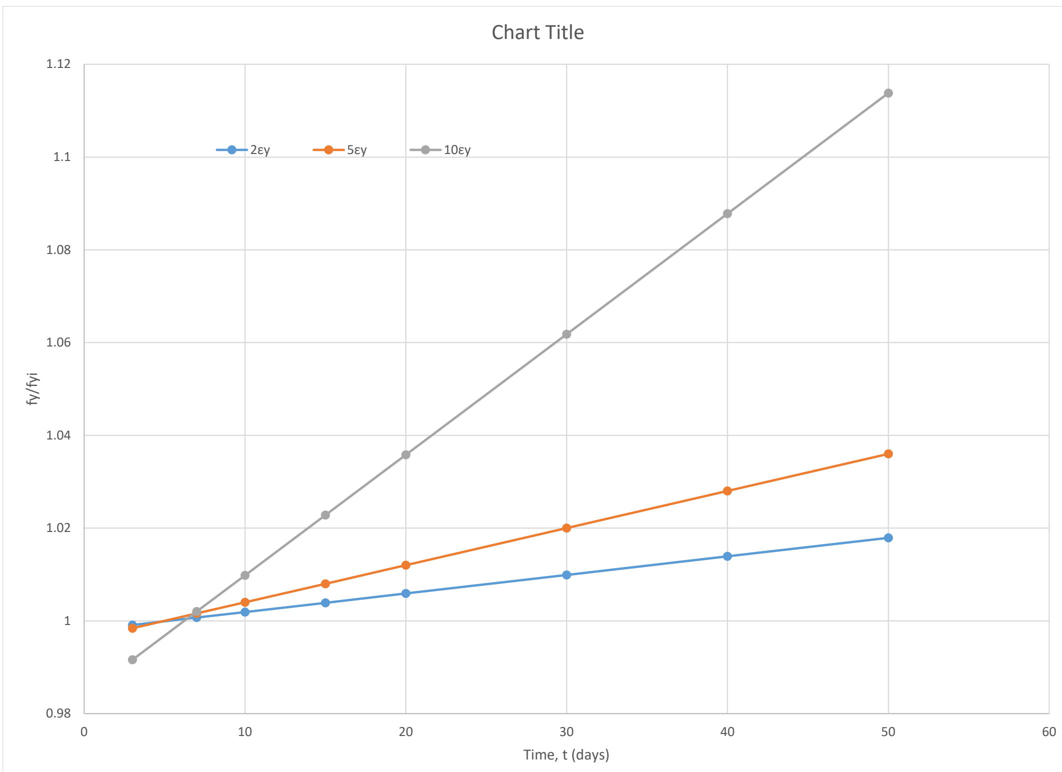

Figure 4.12 Strain Aging effect on Yield Strength vs Time (days)

4.3 Multiple Seismic Events

4.3.1 Main Shock Series

4.3.2 Main Shock - After Shock Series

4.4 Future Topics

- Concrete Strength Aging
- Welding and Fatigue in Steel Structures
- Repair Effects
- Main Shock - After Shock Series - Repair Series

Chapter 5

Analytical Model and Preliminary Results

5.1 SDOF Model

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5.2 Model Calibration

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5.3 Comparison with existing physical Tests

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5.4 Earthquake selection

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5.5 Results from NLTHA

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5.5.1 Effect on global response

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5.5.2 Effect on local response

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5.5.3 Preliminary results

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APPENDIX

Appendix A

LOREM IPSUM

A.1 A First Section

A.1.0.0.1 Filler Text

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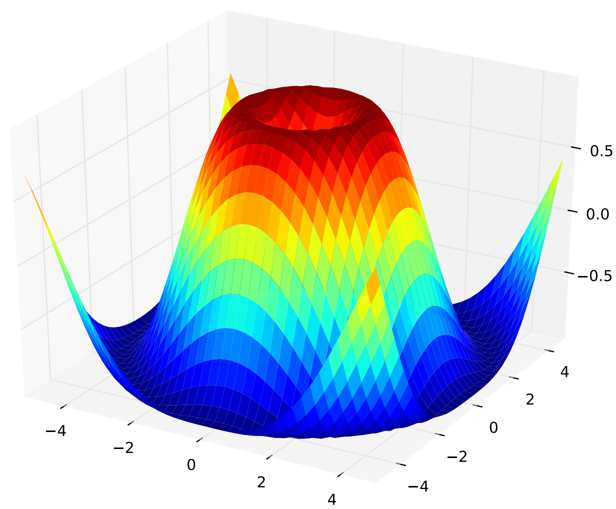

Figure A.1 A figure in the appendix.

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Table A.1 A table in the appendix.

System	Author
TEX	Donald Knuth
LATEX	Leslie Lamport

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A.2 A Second Section

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