ABSTRACT

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

Lorem ipsum dolor sit amet, consectetuer adipiscing elit. Ut purus elit, vestibulum ut, placerat ac, adipiscing vitae, felis. Curabitur dictum gravida mauris. Nam arcu libero, nonummy eget, consectetuer id, vulputate a, magna. Donec vehicula augue eu neque. Pellentesque habitant morbi tristique senectus et netus et malesuada fames ac turpis egestas. Mauris ut leo. Cras viverra metus rhoncus sem. Nulla et lectus vestibulum urna fringilla ultrices. Phasellus eu tellus sit amet tortor gravida placerat. Integer sapien est, iaculis in, pretium quis, viverra ac, nunc. Praesent eget sem vel leo ultrices bibendum. Aenean faucibus. Morbi dolor nulla, malesuada eu, pulvinar at, mollis ac, nulla. Curabitur auctor semper nulla. Donec varius orci eget risus. Duis nibh mi, congue eu, accumsan eleifend, sagittis quis, diam. Duis eget orci sit amet orci dignissim rutrum.

Nam dui ligula, fringilla a, euismod sodales, sollicitudin vel, wisi. Morbi auctor lorem non justo. Nam lacus libero, pretium at, lobortis vitae, ultricies et, tellus. Donec aliquet, tortor sed accumsan bibendum, erat ligula aliquet magna, vitae ornare odio metus a mi. Morbi ac orci et nisl hendrerit mollis. Suspendisse ut massa. Cras nec ante. Pellentesque a nulla. Cum sociis natoque penatibus et magnis dis parturient montes, nascetur ridiculus mus. Aliquam tincidunt urna. Nulla ullamcorper vestibulum turpis. Pellentesque cursus luctus mauris.

Nulla malesuada porttitor diam. Donec felis erat, congue non, volutpat at, tincidunt tristique, libero. Vivamus viverra fermentum felis. Donec nonummy pellentesque ante. Phasellus adipiscing semper elit. Proin fermentum massa ac quam. Sed diam turpis, molestie vitae, placerat a, molestie nec, leo. Maecenas lacinia. Nam ipsum ligula, eleifend at, accumsan nec, suscipit a, ipsum. Morbi blandit ligula feugiat magna. Nunc eleifend consequat lorem. Sed lacinia nulla vitae enim. Pellentesque tincidunt purus vel magna. Integer non enim. Praesent euismod nunc eu purus. Donec bibendum quam in tellus. Nullam cursus pulvinar lectus. Donec et mi. Nam vulputate metus eu enim. Vestibulum pellentesque felis eu massa.

Quisque ullamcorper placerat ipsum. Cras nibh. Morbi vel justo vitae lacus tincidunt ultrices. Lorem ipsum dolor sit amet, consectetuer adipiscing elit. In hac habitasse platea dictumst. Integer tempus convallis augue. Etiam facilisis. Nunc elementum fermentum wisi. Aenean placerat. Ut imperdiet, enim sed gravida sollicitudin, felis odio placerat quam, ac pulvinar elit purus eget enim. Nunc vitae tortor. Proin tempus nibh sit amet nisl. Vivamus quis tortor vitae risus porta vehicula.

Fusce mauris. Vestibulum luctus nibh at lectus. Sed bibendum, nulla a faucibus semper, leo velit ultricies tellus, ac venenatis arcu wisi vel nisl. Vestibulum diam. Aliquam pellentesque,

augue quis sagittis posuere, turpis lacus congue quam, in hendrerit risus eros eget felis. Maecenas eget erat in sapien mattis porttitor. Vestibulum porttitor. Nulla facilisi. Sed a turpis eu lacus commodo facilisis. Morbi fringilla, wisi in dignissim interdum, justo lectus sagittis dui, et vehicula libero dui cursus dui. Mauris tempor ligula sed lacus. Duis cursus enim ut augue. Cras ac magna. Cras nulla. Nulla egestas. Curabitur a leo. Quisque egestas wisi eget nunc. Nam feugiat lacus vel est. Curabitur consectetuer.

Suspendisse vel felis. Ut lorem lorem, interdum eu, tincidunt sit amet, laoreet vitae, arcu. Aenean faucibus pede eu ante. Praesent enim elit, rutrum at, molestie non, nonummy vel, nisl. Ut lectus eros, malesuada sit amet, fermentum eu, sodales cursus, magna. Donec eu purus. Quisque vehicula, urna sed ultricies auctor, pede lorem egestas dui, et convallis elit erat sed nulla. Donec luctus. Curabitur et nunc. Aliquam dolor odio, commodo pretium, ultricies non, pharetra in, velit. Integer arcu est, nonummy in, fermentum faucibus, egestas vel, odio.

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

Raleigh, North Carolina

2019

APPROVED BY:

Dr. James Nau	Dr. Mohammad Pour-Ghaz

Dr. Rudolf Seracino	Dr. Thomas Birkland

Dr. Mervyn Kowalsky
Chair of Advisory Committee

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

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

Lorem ipsum dolor sit amet, consectetuer adipiscing elit. Ut purus elit, vestibulum ut, placerat ac, adipiscing vitae, felis. Curabitur dictum gravida mauris. Nam arcu libero, nonummy eget, consectetuer id, vulputate a, magna. Donec vehicula augue eu neque. Pellentesque habitant morbi tristique senectus et netus et malesuada fames ac turpis egestas. Mauris ut leo. Cras viverra metus rhoncus sem. Nulla et lectus vestibulum urna fringilla ultrices. Phasellus eu tellus sit amet tortor gravida placerat. Integer sapien est, iaculis in, pretium quis, viverra ac, nunc. Praesent eget sem vel leo ultrices bibendum. Aenean faucibus. Morbi dolor nulla, malesuada eu, pulvinar at, mollis ac, nulla. Curabitur auctor semper nulla. Donec varius orci eget risus. Duis nibh mi, congue eu, accumsan eleifend, sagittis quis, diam. Duis eget orci sit amet orci dignissim rutrum.

1.2 Scope and layout

Nam dui ligula, fringilla a, euismod sodales, sollicitudin vel, wisi. Morbi auctor lorem non justo. Nam lacus libero, pretium at, lobortis vitae, ultricies et, tellus. Donec aliquet, tortor sed accumsan bibendum, erat ligula aliquet magna, vitae ornare odio metus a mi. Morbi ac orci et nisl hendrerit mollis. Suspendisse ut massa. Cras nec ante. Pellentesque a nulla. Cum sociis natoque penatibus et magnis dis parturient montes, nascetur ridiculus mus. Aliquam tincidunt urna. Nulla ullamcorper vestibulum turpis. Pellentesque cursus luctus mauris.

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

Lorem ipsum dolor sit amet, consectetuer adipiscing elit. Ut purus elit, vestibulum ut, placerat ac, adipiscing vitae, felis. Curabitur dictum gravida mauris. Nam arcu libero, nonummy eget, consectetuer id, vulputate a, magna. Donec vehicula augue eu neque. Pellentesque habitant morbi tristique senectus et netus et malesuada fames ac turpis egestas. Mauris ut leo. Cras viverra metus rhoncus sem. Nulla et lectus vestibulum urna fringilla ultrices. Phasellus eu tellus sit amet tortor gravida placerat. Integer sapien est, iaculis in, pretium quis, viverra ac, nunc. Praesent eget sem vel leo ultrices bibendum. Aenean faucibus. Morbi dolor nulla, malesuada eu, pulvinar at, mollis ac, nulla. Curabitur auctor semper nulla. Donec varius orci eget risus. Duis nibh mi, congue eu, accumsan eleifend, sagittis quis, diam. Duis eget orci sit amet orci dignissim rutrum.

Nam dui ligula, fringilla a, euismod sodales, sollicitudin vel, wisi. Morbi auctor lorem non justo. Nam lacus libero, pretium at, lobortis vitae, ultricies et, tellus. Donec aliquet, tortor sed accumsan bibendum, erat ligula aliquet magna, vitae ornare odio metus a mi. Morbi ac orci et nisl hendrerit mollis. Suspendisse ut massa. Cras nec ante. Pellentesque a nulla. Cum sociis natoque penatibus et magnis dis parturient montes, nascetur ridiculus mus. Aliquam tincidunt urna. Nulla ullamcorper vestibulum turpis. Pellentesque cursus luctus mauris.

Nulla malesuada porttitor diam. Donec felis erat, congue non, volutpat at, tincidunt tris-

Figure 3.1 A figure at the top of the page.
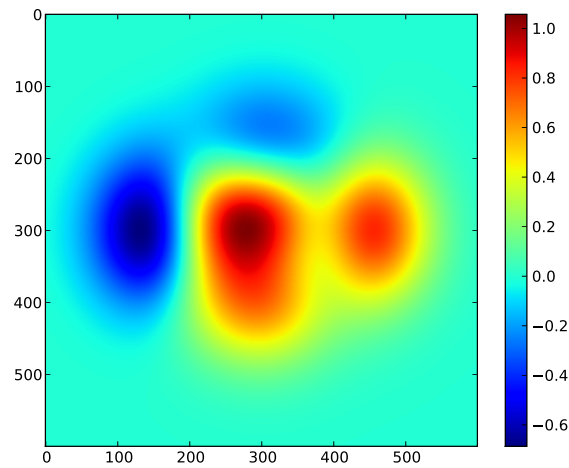
Figure 3.2 A figure in the middle of text.

tique, libero. Vivamus viverra fermentum felis. Donec nonummy pellentesque ante. Phasellus adipiscing semper elit. Proin fermentum massa ac quam. Sed diam turpis, molestie vitae, placerat a, molestie nec, leo. Maecenas lacinia. Nam ipsum ligula, eleifend at, accumsan nec, suscipit a, ipsum. Morbi blandit ligula feugiat magna. Nunc eleifend consequat lorem. Sed lacinia nulla vitae enim. Pellentesque tincidunt purus vel magna. Integer non enim. Praesent euismod nunc

5

eu purus. Donec bibendum quam in tellus. Nullam cursus pulvinar lectus. Donec et mi. Nam vulputate metus eu enim. Vestibulum pellentesque felis eu massa.
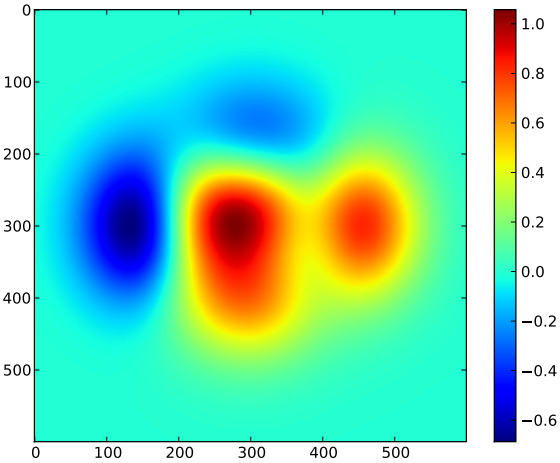
Figure 3.3 A figure at the bottom of the page.

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

These effects generally affect the mechanical behavior of the materials, which are not considered when designing a structure. In the following paragraphs 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 it is the one that is assumed in the present study.

Corrosion of steel in concrete is an electrochemical process [14] this corrosion may 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.

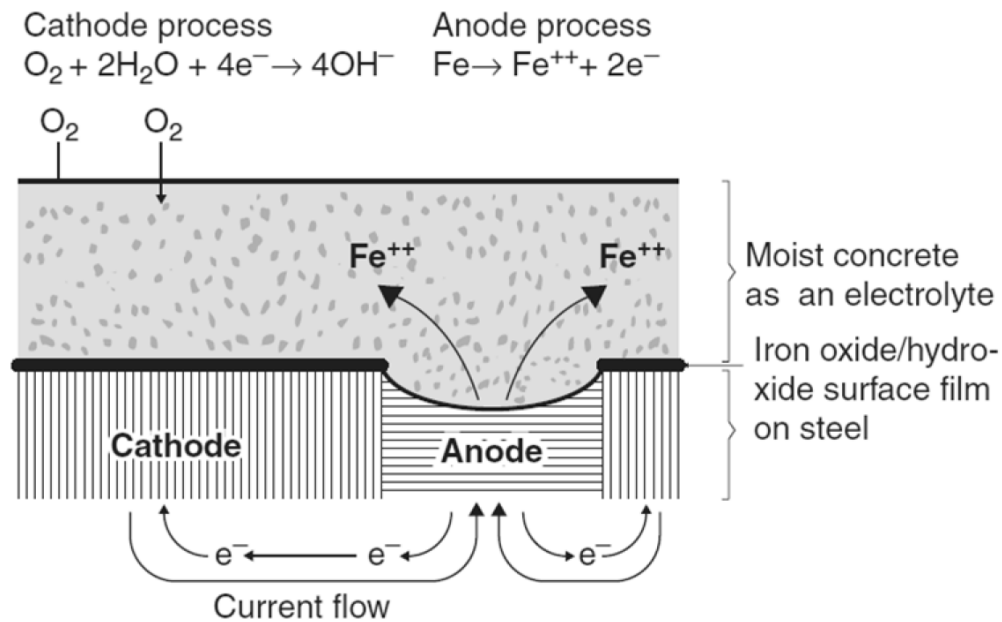**Figure 4.1** Corrosion Process in Reinforcing Steel Bar [14]

A literature review to characterize corrosion in reinforcing steel is presented to model corrosion can be modeled, the corrosion process is an extensive field of research and to accurately incorporate it into the analysis it is necessary to incorporate the following components:

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 [19] 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

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, however, 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 [7]
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 mode since the program is in a closed format.

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:

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

10

$$T_{cr} = \frac{W_{crit}^2}{2k_p} \qquad (4.5)$$

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

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, however, it is able 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 [20], 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.

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

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.
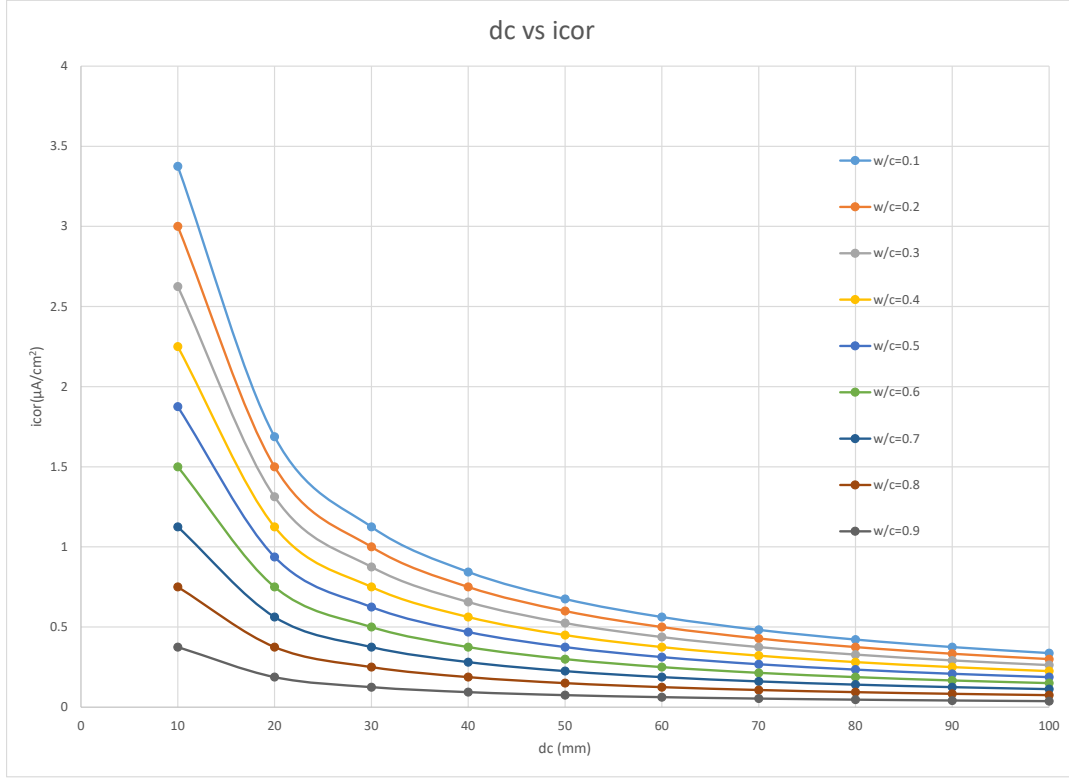
Figure 4.2 Concrete cover depth vs rate of corrosion

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:

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

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

12

Figure 4.3 Diameter decrease due to corrosion

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 [22] it was shown from experimental results that the mechanical properties of steel for different levels of corrosion could be modified for analysis as follows:

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

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

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

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

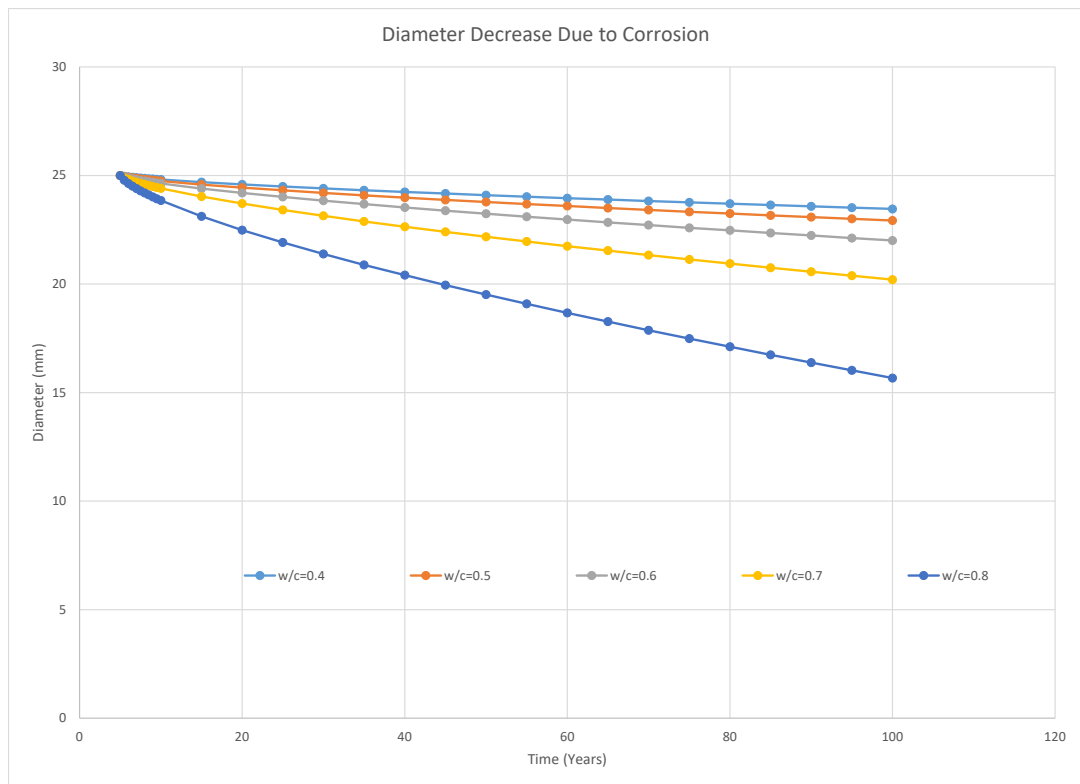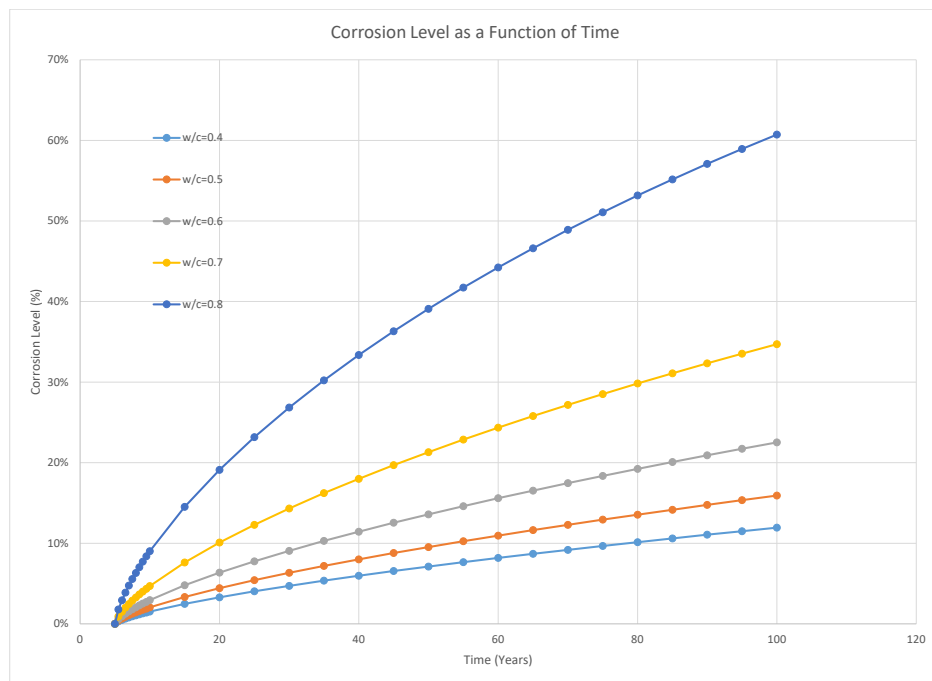**Choe et al. Model**   Choe et al research is a seismic fragility estimates for RC columns subjected to corrosion, while the study is probabilistic in nature it defines the reduction in rebar cross section as:

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

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 [10], [13] and [21] 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 Cor-

rosion 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.
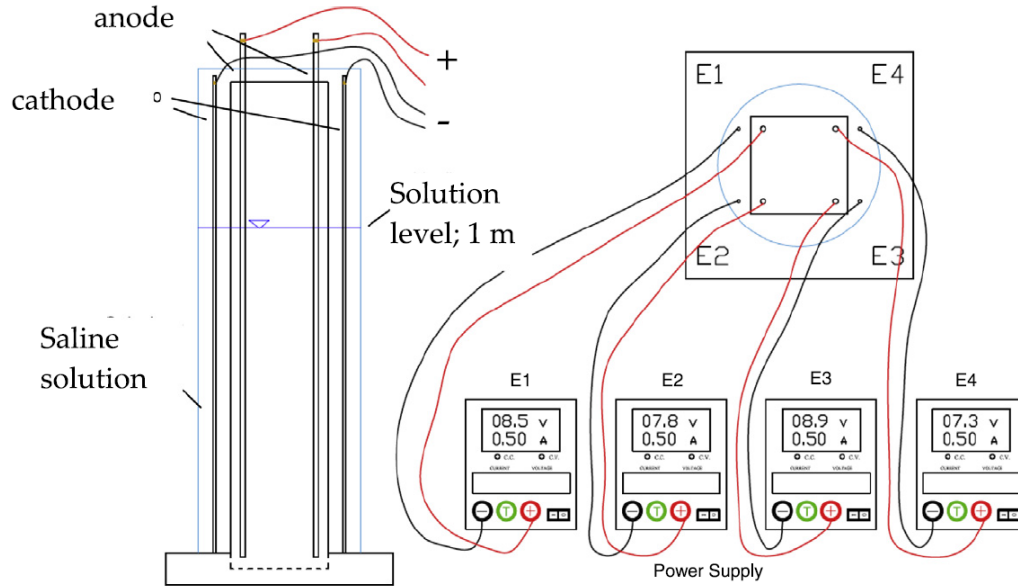
Figure 4.5 Corrosion Process for RC Column [13]

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, in the previous studies [13] 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.

While these studies provided an insight into how corroded RC Columns behave under cyclic loading, they 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
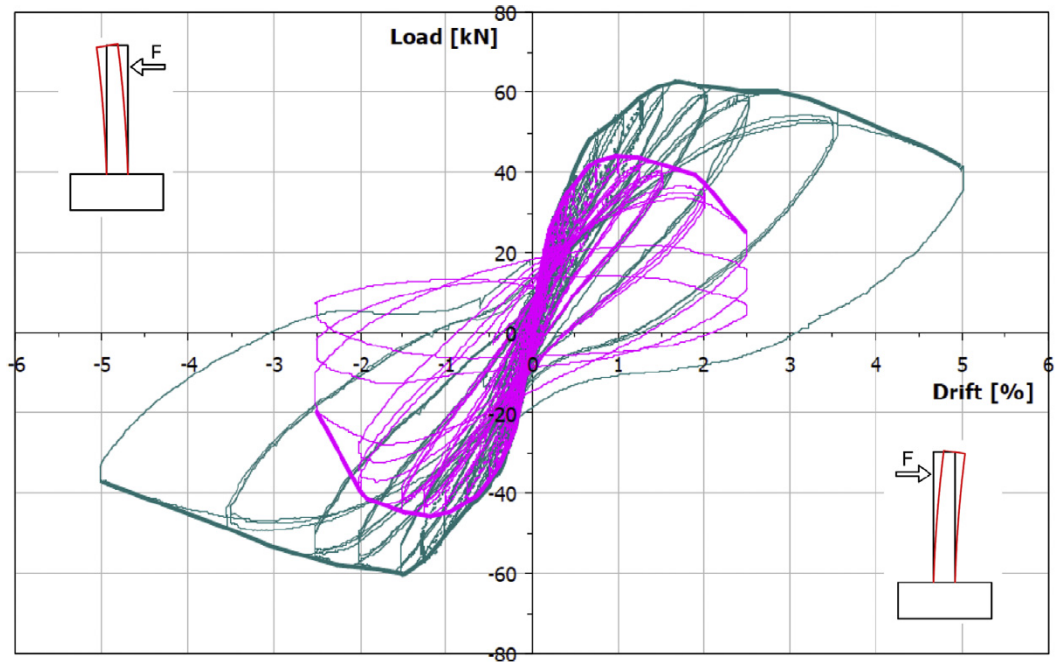
Figure 4.6 Corrosion Process for RC Column [13]

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
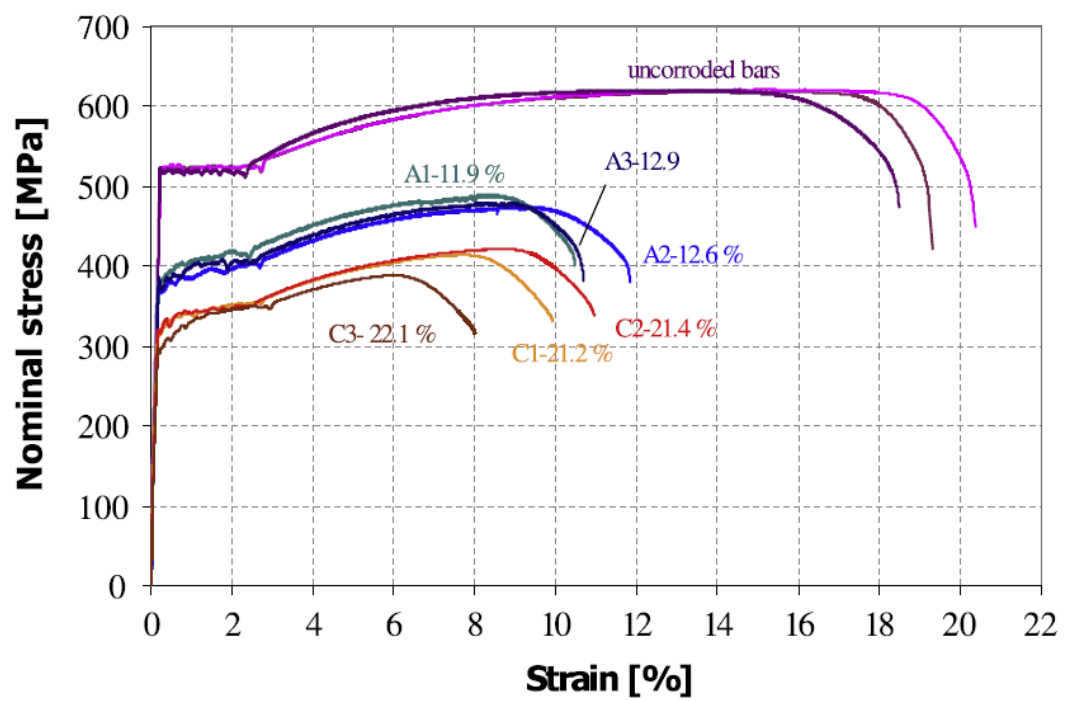
Figure 4.7 Corroded Rebars Stress-Strain Curves [13]

Methods to generate the passive film on reinforcing steel are available in the literature [6]. 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:

- Saturated Calcium Hydroxide $Ca(OH)_2$

- Sodium Hydroxide $Na(OH)$ 4.00 g/l

- Potassium Hydroxide (OH) 11.22 g/l

- Calcium Sulfate Dihydrate $Ca(SO)_4 + 2H_2O$ 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.
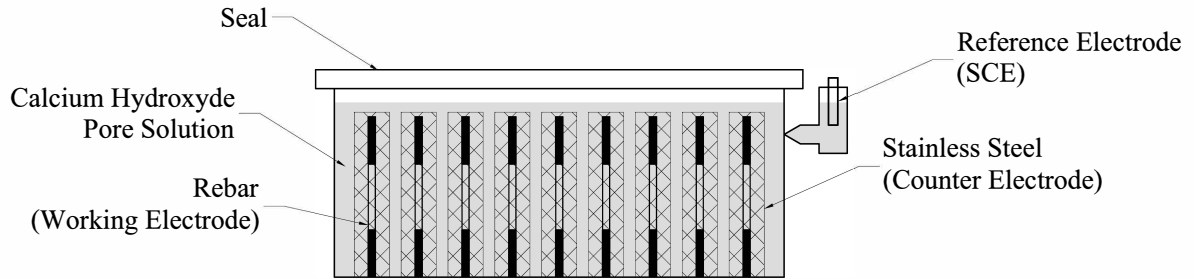
Figure 4.8 Rebars Passivation Process in Calcium Hydroxyde Pore Solution

Accelerated corrosion of Reinforcing Steel

The accelerated corrosion will be done by using a galvanic cell. Different studies [6] 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:
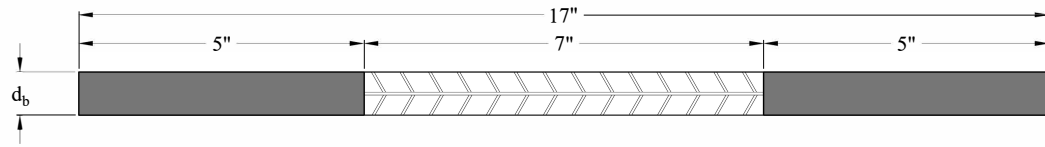
Figure 4.9 Rebar Specimen Geometry
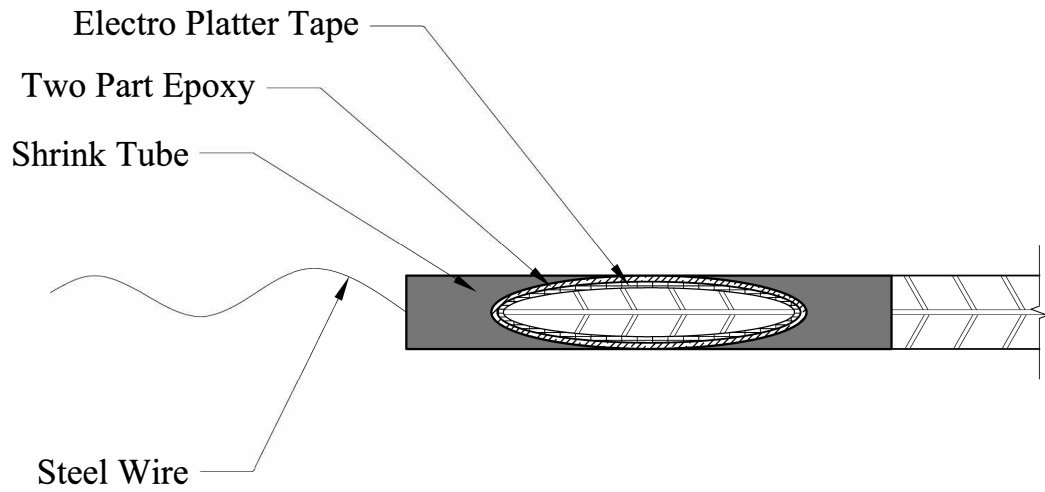
Figure 4.10 Rebars Ends Protection

$$t = \frac{\lambda m_{loss} \eta_{specimen} C_{faraday}}{i M_{specimen}} \qquad (4.14)$$
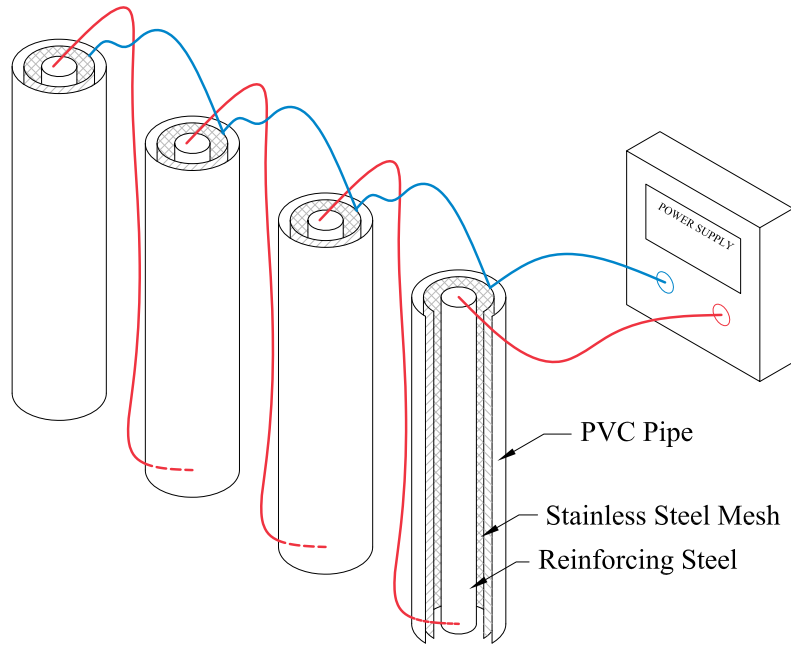
Figure 4.11 Accelerated Corrosion Process

For the different rebar sizes and Corrosion levels the current and the Time of Application is shown in Table 4.1. A current of $5mA$ is applied to the specimen obtaining the time this current needs to be applied for.

Tension Tests

Tension tests will be performed according to ASTM A706. 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 critical bending strain of buckling of reinforced steel [2]. The premise of the BBT Test is a material test to simulate bending and

Table 4.1 Accelerated Corrosion to achieve Corrosion Levels.

Corrosion Level (CL)	Mass loss (g)	time(days)
5%	1.12	9
10%	2.24	18
15%	3.36	27
20%	4.47	36
25%	5.59	45

tension strain demands on a buckled bar.However those results 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:

1. Compress a rebar specimen up to a certain level of compression strain such that the rebar will show buckling

2. The rebar is then pulled untill rupture

3. process is repeated for different levels of compression strains

This test is proposed for different levels of corrosion such that any changes on the behavior are studied and incorporated into the analysis a sequence of the test procedure is shown in Fig. 4.12. A proposed test matrix is shown in Table 4.2.

4.1.6 Modeling of corrosion for Structural Analysis

The previous elements of corrosion explained in the previous sections are incorporated into the structural analysis mainly at the material level. The application can be outlined as follows:

1. First the time for initiation of corrosion is calculated according to the Gosh and Padgett Model [7]

2. Then the rate of corrosion is calculated according to the Vu et al model [20]

3. Following this the size of the rebar is reduced and the corrosion level is calculated

4. Finally the mechanical properties of the reinforcing steel are modified with the corresponding corrosion level.
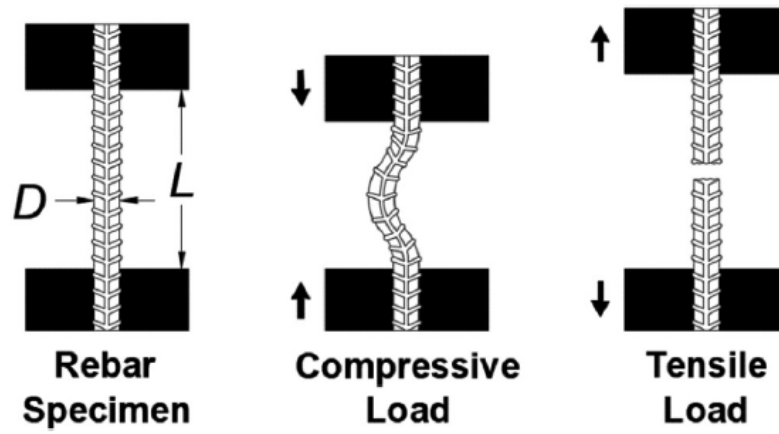
Figure 4.12 BBT Test sequence

Table 4.2 Corroded Rebar Test Matrix

Corroded BBT Test Matrix			
Test	Diameter of Bar	CL (%)	Number of Tests
Tension Test	#6	0	3
		5	3
		10	3
		15	3
		20	3
		25	3
BBT Test	#6	0	6
		5	6
		10	6
		15	6
		20	6
		25	6
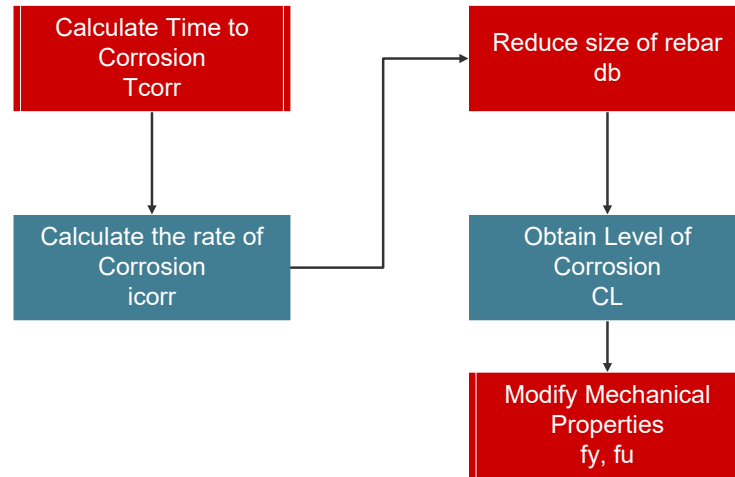
Figure 4.13 Corrosion Modeling for Structural Analysis

This modeling can be better seen in Fig. 4.13

This is later incorporated into the Nonlinear Structural Analysis Framework using the package OpenSees [12], the framework of this analysis is explained in [**Chapter-5**].

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 [17] most strain aging occurs in the first 37 days. Also [15] 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[15].

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 $5\varepsilon_y$

$$\frac{f_y}{f_{yi}} = 0.0008t + 0.996 \tag{4.16}$$
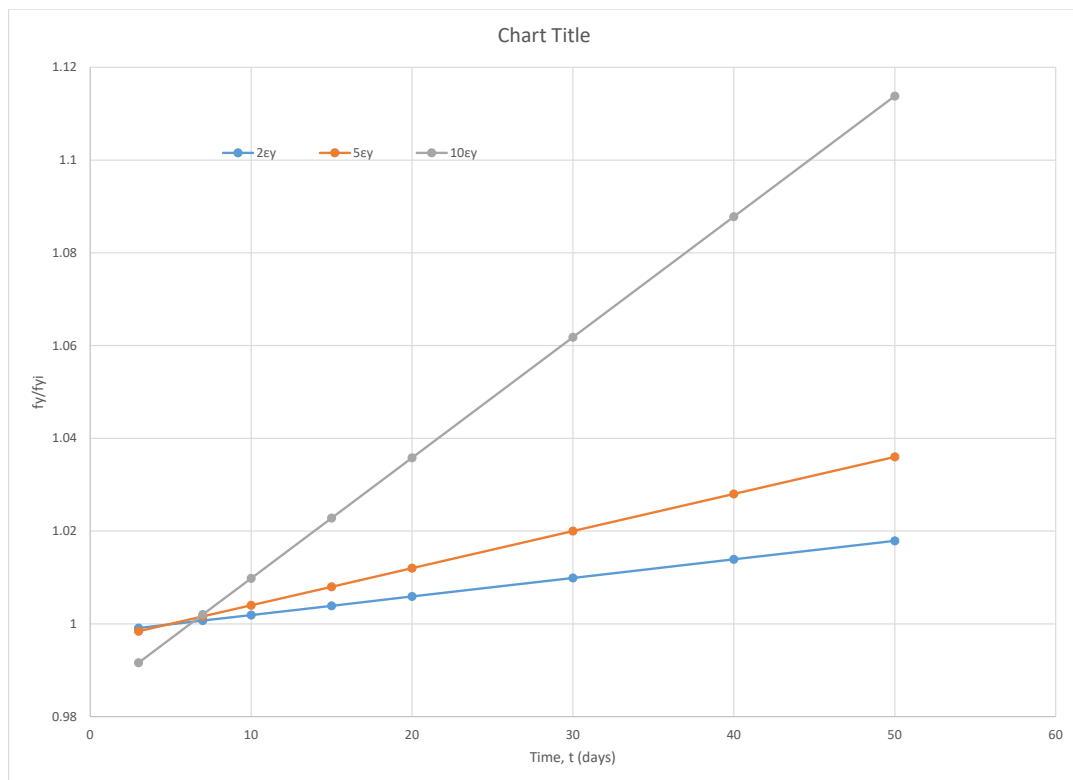
For $2\varepsilon_y$

Figure 4.14 Strain Aging effect on Yield Strength vs Time (days)

$$\frac{f_y}{f_{yi}} = 0.0004t + 0.9979 \qquad (4.17)$$

It is proposed to limit the increase in yield strength to the one obtained at 50 days. These equations are plotted in Fig. 4.14

4.3 Multiple Seismic Events

The evaluation of multiple seismic events is a topic that has been scarcely studied, however their effects have been felt in numerous earthquake sequences such as El Salvador, North Ridge, Chi-Chi among others. The main thought is that after a series of earthquake the structures accumulate damage and would eventually fail, this has been attempted as it was shown in [**Chapter-1**].

For this study it has been determined that not all damage in structures are dependent on multiple events but rather their condition when an event occours as is the case for corrosion. Other damage related phenomenons such as Strain Aging depend on the loading history and are therefore dependent on the history of extreme loading events. It is therefore proposed to study corrosion on a discrete modeling of Main Shocks each independent of the other and to study the effect on Strain Aging by using a sequence of Main Shocks.

4.3.1 Earthquake Selection

For this study the NGA2 West Database of earthquake records provided by the Pacific Earthquake and Engineering Research Insitute (PEER) [1] is used. This database consists of 599 different Earthquake events that characterize theground motions on the west coast of the contiguous United States. The data was filteres to according to the following criteria:

- Earthquake sequence

- Moment Magnitude $M_w \geqslant 5$

- $PGA > 0.04$

- $PGV > 1$ cm/s

- $Vs_{30} > 100m/s$ & $Vs_{30} < 1000$ m/s

- Lowest usable frequency is less than 1Hz

- $R_{rup} < 60km$

From this data the major earthquakes found are the following, the earthquakes can be sumarized in Fig. 4.15 which show this earthquakes as moment magnitude Mw vs rupture distance (R_{rup}).

- Chi-chi

- Managua

- Livermore

- Northridge

- Duzce

- Mammoth lake
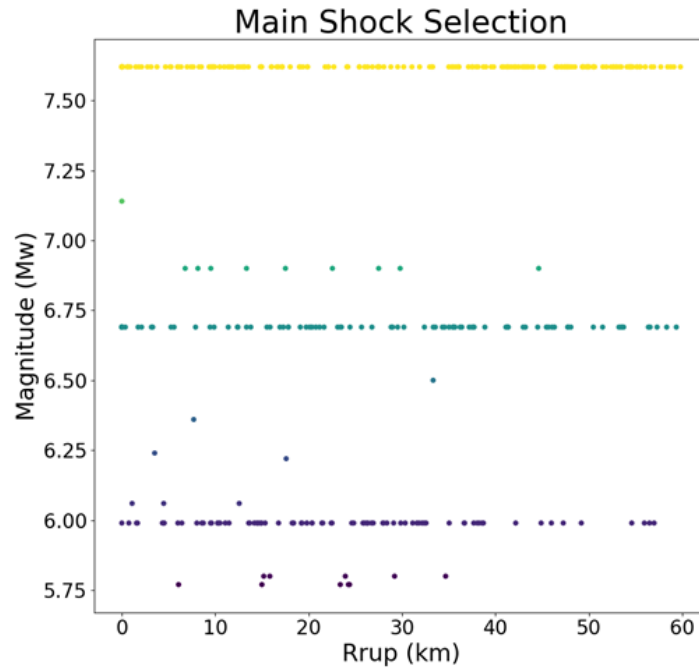
Figure 4.15 Mainshock selection from PEER NGA West2 Database

4.3.2 Discrete Modeling of Main Shock Series

The discrete modeling of mainshocks consists in using individual earthquakes that accour at different times thorughout the life of the structures which correlate to a Corrosion Level (CL), this can be done for each of the main shocks selected after which the follogin data is obtained and later analyzed:

- Maximum axial strain in Confined Concrete, Cover and Reinforcing steel Strains

- Obtain the probability of exceeding a given limit state $P(\varepsilon > \varepsilon_{LS}, IM)$

- The earthquakes are characterized according to an intensity measure

4.3.3 Multiple Main Shock Series

To simulate the life of a structure a Mainshowck series consisting of 3 Mainshocks for a the life of a structure is considered, three phases are considered:

1. at time $t = 0$ the structure has pristine conditions

2. Mainshock 1 occours

3. Mainshock 2 occours

4. Mainshock 3 occours

This is shown graphically in Fig. 4.16
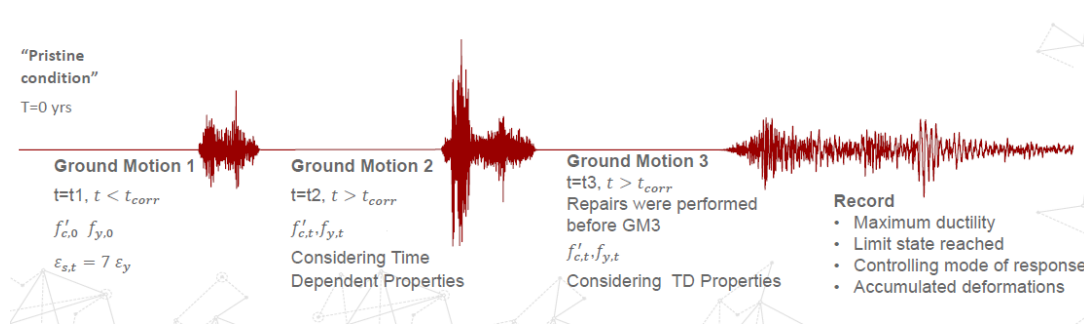
Figure 4.16 Mainshock sequence example

Then from the analysis the same results can be recorded

- Maximum axial strain in Confined Concrete, Cover and Reinforcing steel Strains

- Obtain the probability of exceeding a given limit state $P(\varepsilon\varepsilon_{LS}, IM)$

- The earthquakes are characterized according to an intensity measure

4.4   Future Topics

- Concrete Strength Aging

- Welding and Fatigue in Steel Structures

- Repair Effects

- Main Shock - After Shock Series - Repair Series

- Degree of damage effect on confined structures behavior

Chapter 5

Analytical Model and Preliminary Results

In this chapter first a framework for the analysis that is performed is presented, later the basic model that is used for the analysis is presented and later calibrated and verified with experimental data available in the literature. Finally preliminary results are presented that will define the general view for the proposed research

5.1 Analytical Model

5.1.1 Cantilever Column

This study focuses on the behavior of a Single Degree of Freedom System representing a Cantilever Reinforced Concrete Column. The column is modeled as shown in Fig. 5.1 This structure is modeled in OpenSeesPy [12][24] using the $forceBeamColumn$ element [18]. The forceBeamColumn element is used with two-point Gauss-Radau integration applied in the hinge regions and two-point Gauss integration applied on the element interior for a total of six integration points [18]. The force based formulation requires only a single element to accurately represent the full nonlinear deformation of the member and the integration scheme selected prevents the loss of objectivity during softening response while also providing integration points at the member ends [3],[18]. The element requires the length of plasticity be defined at each end of the member, for which the tension based rectangular plastic hinge length is calculated using the following expressions [8]:

$$L_{pc} = k * L_{eff} + 0.4D \tag{5.1}$$

$$k = 0.2 * (Fu/Fy - 1) \leqslant 0.08 \tag{5.2}$$

$$L_{pt} = L_{pc} + \gamma * D \tag{5.3}$$

For Single Bending

$$\gamma = 0.33 \tag{5.4}$$

The two-point Gauss-Radau integration is applied such that each end node integration is weighted equal to the specified plastic hinge length, as illustrated in Fig. 5.2. Therefore, strains recorded at the end sections represent accurate values even in the case where deformation localizes to the ends from strain softening behavior. For the case of the cantilever column considered, only one plastic hinge length is defined, and the opposite end is given an arbitrary unit length.

The section of the column is shown in Fig. 5.3, the section is discretized with concrete and steel material fibers. Concrete fibers are modeled using the $Concrete01$ material, modified for confined material strength based on the Mander confined concrete model [11]. The $Steel02$ material, based on the Giuffre-Menegotto-Pinto model [5] and it is used for the longitudinal reinforcement with recommended parameters ($b = 0.01, R0 = 20, cR1 = 0.925, cR2 = 0.15$).

5.1.2 Strain Penetration Component

The strain penetration is necessary to be considered to take into account the additional deformation due to anchorage of the reinforcement into the foundation, since the strains of tension in the reinforcement will drop to zero at a depth equal to the true development length of the rebar [16]. Experimental studies have generally reported that this end rotation contributes up to 35% to the lateral deformation of flexural members[23] and it is therefore important to incorporate into the analytical model. A way to capture this effect is by using a zero-length section element implemented in nonlinear fiber-based analysis of concrete structures, this is available in the material library of OpenSeesPy as $BondSP1$ [23] this is material model used for the steel fibers of the zero-length section element.

The required parameters for this model are:

- F_y Yield strength of the reinforcement steel

- S_y Rebar slip at member interface under yield stress (see Eq. 5.5)
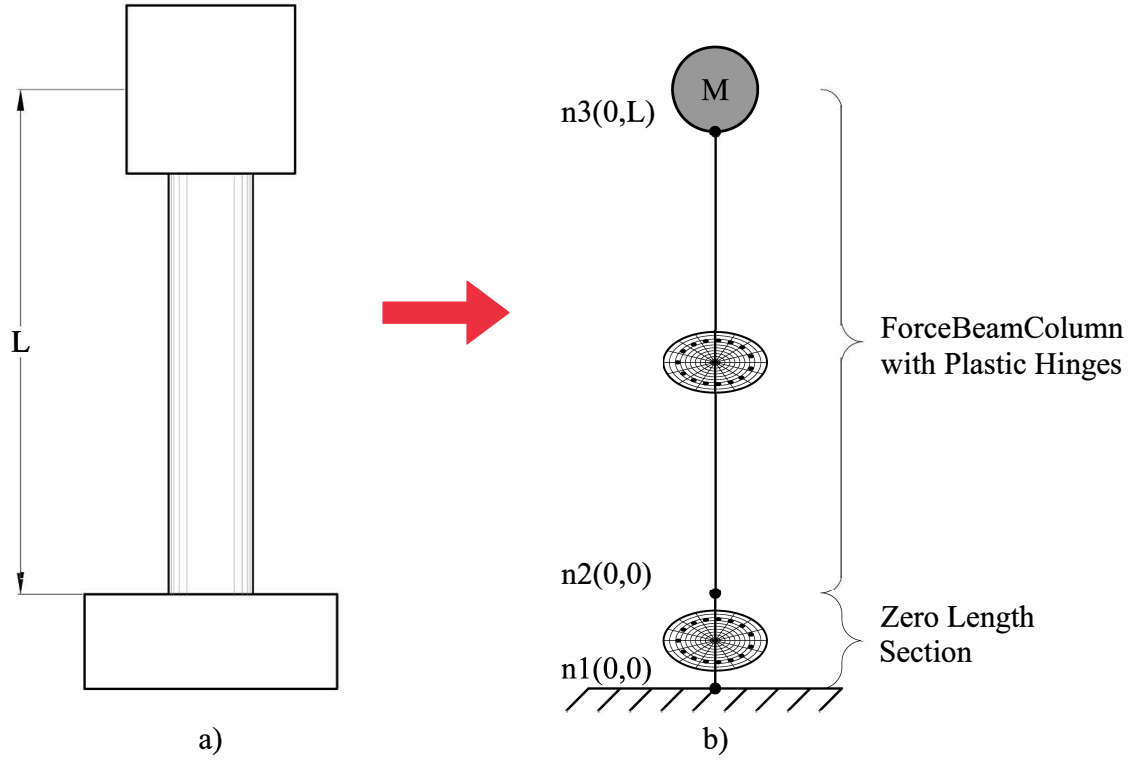
Figure 5.1 Structural Model a) SDOF Column b) Structural Model
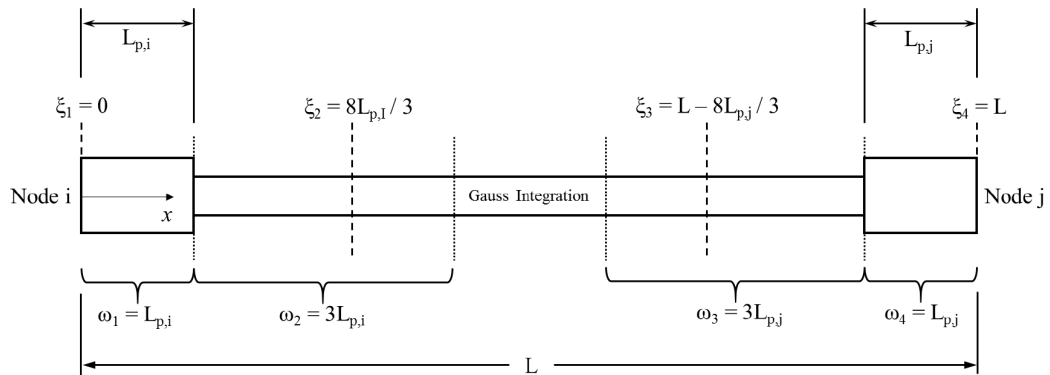
Figure 5.2 End point plastic hinge method [18]
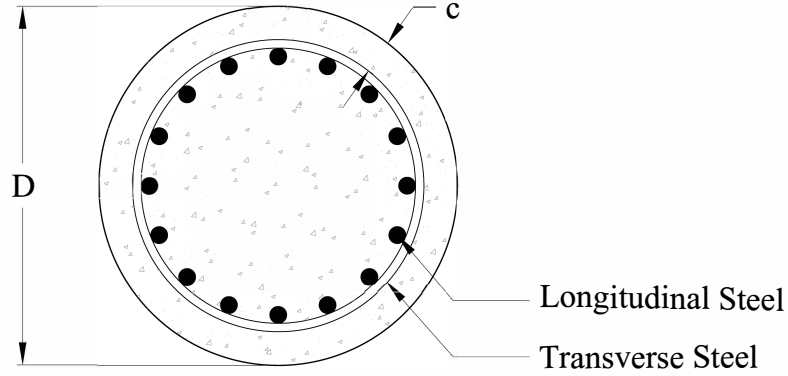
Figure 5.3 Section of the RC Column

- F_u Ultimate strength of the reinforcement steel

- S_u Rebar slip at the loaded end at the bar fracture strength a value of $35S_y$ is recommended [23]

- b Initial hardening ratio in the monotonic slip vs. bar stress response $b = 0.45$ is recommended [23]

- R Pinching factor for the cyclic slip vs. bar response $R = 1.01$ is recommended [23]

- d_b Rebar diameter

- $f'c$ Concrete compressive strength of the adjoining connection member

- α Parameter used in the local bond-slip relation and can be taken as $\alpha = 0.4$ in accordance with CEB-FIP Model Code 90 [4]

Bar slip is calculated as:

$$S_y(in) = 0.1 \left(\frac{d_b F_y}{4000\sqrt{f'_c}} \left(2\alpha + 1\right) \right)^{\frac{1}{\alpha}} + 0.013(in) \tag{5.5}$$

5.1.3   Design Limit States

Design limit states are defined on the basis of strains in the material since they can more accurately represent the different performance level of a structure. Structure limit states are

defined for tension strains in the rebars or compression strains in the concrete core. The values recommended in typical performance based design of reinforced concrete bridge columns are shown in 5.1. The serviceability limit states correspond to the compression strain at which concrete cover begins to crush and the peak tension strain which results in residual crack widths of approximately 1 mm. These limits are generally accepted as nominal limit states for RC members. The compression limit state for damage control is defined by the expresion shown in 5.6 and it refers to the compression strain in the confined concrete at which fracture of the transverse reinforcement confining the core occurs [16]. This equation is obtain via a strain-energy balance between that absorbed by the confined core concrete and the capacity of the confining steel. The tension damage control limit state is defined by the strain at the onset of buckling which can be expressed according to 5.7, this equation demonstrated accurate predictions of the onset of bar buckling on physical tests in SDOF Concrete Column [9].

$$\varepsilon_{c,spiralyield} = 0.009 - 0.3\frac{A_{st}}{A_g} + 3.9\frac{f_{yhe}}{E_s} \tag{5.6}$$

$$\varepsilon_{s,BB} = 0.03 + 700\rho_s\frac{f_{yhe}}{E_s} - 0.1\frac{P}{f'_c A_g} \tag{5.7}$$

Table 5.1 Design Limit States

Limit State	Concrete Limit State $\varepsilon_c(in/in)$	Reinforcing Steel Limit State $\varepsilon_s(in/in)$
Serviciability	0.004	0.015
Damage Control	5.6	5.7

5.2 Comparison with existing physical Tests

5.2.1 Pristine Condition Columns

Nam dui ligula, fringilla a, euismod sodales, sollicitudin vel, wisi. Morbi auctor lorem non justo. Nam lacus libero, pretium at, lobortis vitae, ultricies et, tellus. Donec aliquet, tortor sed accumsan bibendum, erat ligula aliquet magna, vitae ornare odio metus a mi. Morbi ac orci et nisl hendrerit mollis. Suspendisse ut massa. Cras nec ante. Pellentesque a nulla. Cum sociis natoque penatibus et magnis dis parturient montes, nascetur ridiculus mus. Aliquam tincidunt

urna. Nulla ullamcorper vestibulum turpis. Pellentesque cursus luctus mauris.

5.2.2 Accelerated Corrosion Columns

Nulla malesuada porttitor diam. Donec felis erat, congue non, volutpat at, tincidunt tristique, libero. Vivamus viverra fermentum felis. Donec nonummy pellentesque ante. Phasellus adipiscing semper elit. Proin fermentum massa ac quam. Sed diam turpis, molestie vitae, placerat a, molestie nec, leo. Maecenas lacinia. Nam ipsum ligula, eleifend at, accumsan nec, suscipit a, ipsum. Morbi blandit ligula feugiat magna. Nunc eleifend consequat lorem. Sed lacinia nulla vitae enim. Pellentesque tincidunt purus vel magna. Integer non enim. Praesent euismod nunc eu purus. Donec bibendum quam in tellus. Nullam cursus pulvinar lectus. Donec et mi. Nam vulputate metus eu enim. Vestibulum pellentesque felis eu massa.

5.3 Analytical Framework

An overall analytical framework is established such that several analysis can be performed. From this analysis it is possible to determine the effects of damage in the performance of structures. The proposed analytical framework consists in:

1. Geometrical Properties of the SDOF column

2. Properties of the material are evaluated (i.e. water to cement ratio, cover)

3. For equal periods of time the Time Dependent Properties are modified

4. Nonlinear Time History Analysis are performed for discrete events or sequence of events

5. Results are obtained and evaluated

This procedure has been summarized in the form of a flow chart presented in Fig. 5.4

5.4 Earthquake selection

Quisque ullamcorper placerat ipsum. Cras nibh. Morbi vel justo vitae lacus tincidunt ultrices. Lorem ipsum dolor sit amet, consectetuer adipiscing elit. In hac habitasse platea dictumst. Integer tempus convallis augue. Etiam facilisis. Nunc elementum fermentum wisi. Aenean placerat. Ut imperdiet, enim sed gravida sollicitudin, felis odio placerat quam, ac pulvinar elit purus eget enim. Nunc vitae tortor. Proin tempus nibh sit amet nisl. Vivamus quis tortor vitae risus porta vehicula.
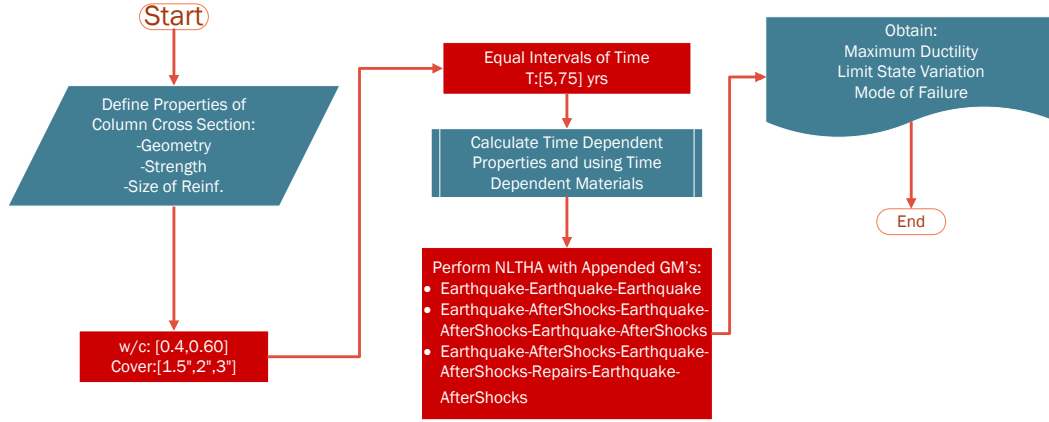
Figure 5.4 Analysis Framework Flowchart

5.5   Results from NLTHA

Fusce mauris. Vestibulum luctus nibh at lectus. Sed bibendum, nulla a faucibus semper, leo velit ultricies tellus, ac venenatis arcu wisi vel nisl. Vestibulum diam. Aliquam pellentesque, augue quis sagittis posuere, turpis lacus congue quam, in hendrerit risus eros eget felis. Maecenas eget erat in sapien mattis porttitor. Vestibulum porttitor. Nulla facilisi. Sed a turpis eu lacus commodo facilisis. Morbi fringilla, wisi in dignissim interdum, justo lectus sagittis dui, et vehicula libero dui cursus dui. Mauris tempor ligula sed lacus. Duis cursus enim ut augue. Cras ac magna. Cras nulla. Nulla egestas. Curabitur a leo. Quisque egestas wisi eget nunc. Nam feugiat lacus vel est. Curabitur consectetuer.

5.5.1   Effect on structure response

Suspendisse vel felis. Ut lorem lorem, interdum eu, tincidunt sit amet, laoreet vitae, arcu. Aenean faucibus pede eu ante. Praesent enim elit, rutrum at, molestie non, nonummy vel, nisl. Ut lectus eros, malesuada sit amet, fermentum eu, sodales cursus, magna. Donec eu purus. Quisque vehicula, urna sed ultricies auctor, pede lorem egestas dui, et convallis elit erat sed nulla. Donec luctus. Curabitur et nunc. Aliquam dolor odio, commodo pretium, ultricies non, pharetra in, velit. Integer arcu est, nonummy in, fermentum faucibus, egestas vel, odio.

5.5.2 Effect on material response

Sed commodo posuere pede. Mauris ut est. Ut quis purus. Sed ac odio. Sed vehicula hendrerit sem. Duis non odio. Morbi ut dui. Sed accumsan risus eget odio. In hac habitasse platea dictumst. Pellentesque non elit. Fusce sed justo eu urna porta tincidunt. Mauris felis odio, sollicitudin sed, volutpat a, ornare ac, erat. Morbi quis dolor. Donec pellentesque, erat ac sagittis semper, nunc dui lobortis purus, quis congue purus metus ultricies tellus. Proin et quam. Class aptent taciti sociosqu ad litora torquent per conubia nostra, per inceptos hymenaeos. Praesent sapien turpis, fermentum vel, eleifend faucibus, vehicula eu, lacus.

5.5.3 Preliminary results

Pellentesque habitant morbi tristique senectus et netus et malesuada fames ac turpis egestas. Donec odio elit, dictum in, hendrerit sit amet, egestas sed, leo. Praesent feugiat sapien aliquet odio. Integer vitae justo. Aliquam vestibulum fringilla lorem. Sed neque lectus, consectetuer at, consectetuer sed, eleifend ac, lectus. Nulla facilisi. Pellentesque eget lectus. Proin eu metus. Sed porttitor. In hac habitasse platea dictumst. Suspendisse eu lectus. Ut mi mi, lacinia sit amet, placerat et, mollis vitae, dui. Sed ante tellus, tristique ut, iaculis eu, malesuada ac, dui. Mauris nibh leo, facilisis non, adipiscing quis, ultrices a, dui.

BIBLIOGRAPHY

[1] Ancheta, T. D., Darragh, R. B., Stewart, J. P., Seyhan, E., Silva, W. J., S-J Chiou, B., Wooddell, K. E., Graves, R. W., Kottke, A. R., Boore, D. M., Kishida, T. & Donahue, J. L., "NGA-West2 Database," 2014.

[2] Barcley, L. & Kowalsky, M., "Critical Bending Strain of Reinforcing Steel and Buckled Bar Tension Test," *ACI Materials Journal*, **116**, no. 3, 2019.

[3] Calabrese, A., Almeida, J. P. & Pinho, R., "Numerical issues in distributed inelasticity modeling of rc frame elements for seismic analysis," *Journal of Earthquake Engineering*, **14**, no. sup1, pp. 38–68, 2010.

[4] Comité Euro-Intenational du Betón, *CEB-FIP Model Code 90*, 1st. London, 1993.

[5] Filippou, F. C., Popov, E. & Bertero, V., "Effects of bond deterioration on hysteretic behavior of renforce concrete coint," Tech. Rep. August, 1983, p. 212.

[6] Ghods, P., Isgor, O. B., McRae, G. A. & Gu, G. P., "Electrochemical investigation of chloride-induced depassivation of black steel rebar under simulated service conditions," *Corrosion Science*, **52**, no. 5, pp. 1649–1659, 2010.

[7] Ghosh, J. & Padgett, J. E., "Aging Considerations in the Development of Time-Dependent Seismic Fragility Curves," *Journal of Structural Engineering*, **136**, no. 12, pp. 1497–1511, 2010.

[8] Goodnight, J. C., Kowalsky, M. J. & Nau, J. M., "Effect of load history on performance limit states of circular bridge columns," *Journal of Bridge Engineering*, **18**, no. 12, pp. 1383–1396, 2013.

[9] ——, "Strain limit states for circular RC bridge columns," *Earthquake Spectra*, 2016.

[10] Ma, Y., Che, Y. & Gong, J., "Behavior of corrosion damaged circular reinforced concrete columns under cyclic loading," *Construction and Building Materials*, **29**, pp. 548–556, 2012.

[11] Mander, J. B., Priestley, M. J. N. & Park, R., "Theoretical stress‐strain model for confined concrete," *Journal of Structural Engineering*, **114**, no. 8, pp. 1804–1826, 1988.

[12] McKenna, F., Scott, M. H. & Fenves, G. L., "Nonlinear finite-element analysis software architecture using object composition," *Journal of Computing in Civil Engineering*, **24**, no. 1, pp. 95–107, 2010.

[13] Meda, A., Mostosi, S., Rinaldi, Z. & Riva, P., "Experimental evaluation of the corrosion influence on the cyclic behaviour of RC columns," *Engineering Structures*, **76**, pp. 112–123, 2014.

[14] Mehta, P. K. & Monteiro, P. J. M., *Concrete: Microstructure, Properties, and Materials*, 4th ed. McGraw Hill, 2014.

[15] Momtahan, A., Dhakal, R. P. & Rieder, A., "Effects of strain-ageing on New Zealand reinforcing steel bars," *Bulletin of the New Zealand Society for Earthquake Engineering*, **42**, no. 3, pp. 179–186, 2009.

[16] Priestley, M., Calvi, G. M. & Kowalsky, M. J., *Displacement-Based Seismic Design of Structures*, 1st. Pavia, Italy: IUSS Press, 2007.

[17] Restrepo-Posada, J., Dodd, L. L., Park, R & Cooke, N, "Variables Affecting Cyclic Behavior of Reinforcing Steel," *Journal of Structural Engineering*, **120**, no. 11, pp. 3178–3196, 1994.

[18] Scott, M. H. & Fenves, G. L., "Plastic Hinge Integration Methods for Force-Based Beam-Column Elements," *Journal of Structural Engineering*, **132**, no. 2, pp. 244–252, 2006.

[19] Thoft-Christensen, P., "Corrosion and Cracking of Reinforced Concrete,"

[20] Vu, K. A. T. & Stewart, M. G., "Structural reliability of concrete bridges including improved chloride-induced corrosion models," *Structural Safety*, **22**, no. 4, pp. 313–333, 2000.

[21] Yang, S.-Y., Song, X.-B., Jia, H.-X., Chen, X. & Liu, X.-L., "Experimental research on hysteretic behaviors of corroded reinforced concrete columns with different maximum amounts of corrosion of rebar," *Construction and Building Materials*, **121**, pp. 319–327, 2016.

[22] Yuan, Z., Fang, C., Parsaeimaram, M. & Yang, S., "Cyclic Behavior of Corroded Reinforced Concrete Bridge Piers," *Journal of Bridge Engineering*, **22**, no. 7, 2017.

[23] Zhao, J. & Sritharan, S., "Modeling of strain penetration effects in fiber-based analysis of reinforced concrete structures," *ACI Structural Journal*, **104**, no. 2, pp. 133–141, 2007. arXiv: `arXiv:1011.1669v3`.

[24] Zhu, M., McKenna, F. & Scott, M. H., "OpenSeesPy: Python library for the OpenSees finite element framework," *SoftwareX*, **7**, pp. 6–11, 2018.

APPENDIX

Appendix A

LOREM IPSUM

A.1   A First Section

A.1.0.0.1   Filler Text

Lorem ipsum dolor sit amet, consectetuer adipiscing elit. Ut purus elit, vestibulum ut, placerat ac, adipiscing vitae, felis. Curabitur dictum gravida mauris. Nam arcu libero, nonummy eget, consectetuer id, vulputate a, magna. Donec vehicula augue eu neque. Pellentesque habitant morbi tristique senectus et netus et malesuada fames ac turpis egestas. Mauris ut leo. Cras viverra metus rhoncus sem. Nulla et lectus vestibulum urna fringilla ultrices. Phasellus eu tellus sit amet tortor gravida placerat. Integer sapien est, iaculis in, pretium quis, viverra ac, nunc. Praesent eget sem vel leo ultrices bibendum. Aenean faucibus. Morbi dolor nulla, malesuada eu, pulvinar at, mollis ac, nulla. Curabitur auctor semper nulla. Donec varius orci eget risus. Duis nibh mi, congue eu, accumsan eleifend, sagittis quis, diam. Duis eget orci sit amet orci dignissim rutrum.

Nam dui ligula, fringilla a, euismod sodales, sollicitudin vel, wisi. Morbi auctor lorem non justo. Nam lacus libero, pretium at, lobortis vitae, ultricies et, tellus. Donec aliquet, tortor sed accumsan bibendum, erat ligula aliquet magna, vitae ornare odio metus a mi. Morbi ac orci et nisl hendrerit mollis. Suspendisse ut massa. Cras nec ante. Pellentesque a nulla. Cum sociis natoque penatibus et magnis dis parturient montes, nascetur ridiculus mus. Aliquam tincidunt urna. Nulla ullamcorper vestibulum turpis. Pellentesque cursus luctus mauris.

Nulla malesuada porttitor diam. Donec felis erat, congue non, volutpat at, tincidunt tristique, libero. Vivamus viverra fermentum felis. Donec nonummy pellentesque ante. Phasellus adipiscing semper elit. Proin fermentum massa ac quam. Sed diam turpis, molestie vitae, plac-

erat a, molestie nec, leo. Maecenas lacinia. Nam ipsum ligula, eleifend at, accumsan nec, suscipit a, ipsum. Morbi blandit ligula feugiat magna. Nunc eleifend consequat lorem. Sed lacinia nulla vitae enim. Pellentesque tincidunt purus vel magna. Integer non enim. Praesent euismod nunc eu purus. Donec bibendum quam in tellus. Nullam cursus pulvinar lectus. Donec et mi. Nam vulputate metus eu enim. Vestibulum pellentesque felis eu massa.

Quisque ullamcorper placerat ipsum. Cras nibh. Morbi vel justo vitae lacus tincidunt ultrices. Lorem ipsum dolor sit amet, consectetuer adipiscing elit. In hac habitasse platea dictumst. Integer tempus convallis augue. Etiam facilisis. Nunc elementum fermentum wisi. Aenean placerat. Ut imperdiet, enim sed gravida sollicitudin, felis odio placerat quam, ac pulvinar elit purus eget enim. Nunc vitae tortor. Proin tempus nibh sit amet nisl. Vivamus quis tortor vitae risus porta vehicula.

Fusce mauris. Vestibulum luctus nibh at lectus. Sed bibendum, nulla a faucibus semper, leo velit ultricies tellus, ac venenatis arcu wisi vel nisl. Vestibulum diam. Aliquam pellentesque, augue quis sagittis posuere, turpis lacus congue quam, in hendrerit risus eros eget felis. Maecenas eget erat in sapien mattis porttitor. Vestibulum porttitor. Nulla facilisi. Sed a turpis eu lacus commodo facilisis. Morbi fringilla, wisi in dignissim interdum, justo lectus sagittis dui, et vehicula libero dui cursus dui. Mauris tempor ligula sed lacus. Duis cursus enim ut augue. Cras ac magna. Cras nulla. Nulla egestas. Curabitur a leo. Quisque egestas wisi eget nunc. Nam feugiat lacus vel est. Curabitur consectetuer.

Suspendisse vel felis. Ut lorem lorem, interdum eu, tincidunt sit amet, laoreet vitae, arcu. Aenean faucibus pede eu ante. Praesent enim elit, rutrum at, molestie non, nonummy vel, nisl. Ut lectus eros, malesuada sit amet, fermentum eu, sodales cursus, magna. Donec eu purus. Quisque vehicula, urna sed ultricies auctor, pede lorem egestas dui, et convallis elit erat sed nulla. Donec luctus. Curabitur et nunc. Aliquam dolor odio, commodo pretium, ultricies non, pharetra in, velit. Integer arcu est, nonummy in, fermentum faucibus, egestas vel, odio.

Sed commodo posuere pede. Mauris ut est. Ut quis purus. Sed ac odio. Sed vehicula hendrerit sem. Duis non odio. Morbi ut dui. Sed accumsan risus eget odio. In hac habitasse platea dictumst. Pellentesque non elit. Fusce sed justo eu urna porta tincidunt. Mauris felis odio, sollicitudin sed, volutpat a, ornare ac, erat. Morbi quis dolor. Donec pellentesque, erat ac sagittis semper, nunc dui lobortis purus, quis congue purus metus ultricies tellus. Proin et quam. Class aptent taciti sociosqu ad litora torquent per conubia nostra, per inceptos hymenaeos. Praesent sapien turpis, fermentum vel, eleifend faucibus, vehicula eu, lacus.

Pellentesque habitant morbi tristique senectus et netus et malesuada fames ac turpis egestas. Donec odio elit, dictum in, hendrerit sit amet, egestas sed, leo. Praesent feugiat sapien aliquet odio. Integer vitae justo. Aliquam vestibulum fringilla lorem. Sed neque lectus, consectetuer at,
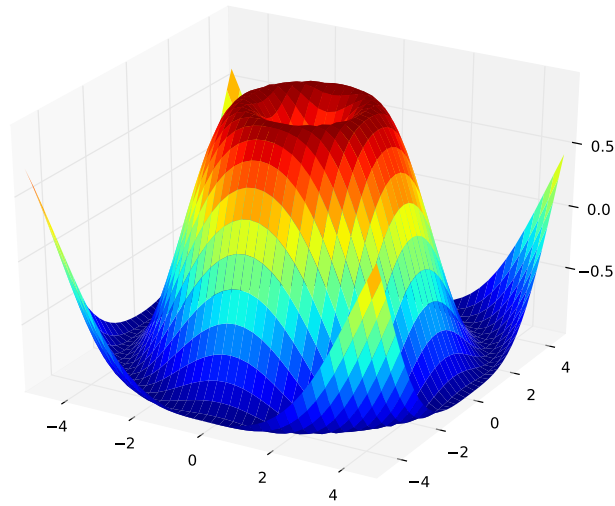
Figure A.1 A figure in the appendix.

consectetuer sed, eleifend ac, lectus. Nulla facilisi. Pellentesque eget lectus. Proin eu metus. Sed porttitor. In hac habitasse platea dictumst. Suspendisse eu lectus. Ut mi mi, lacinia sit amet, placerat et, mollis vitae, dui. Sed ante tellus, tristique ut, iaculis eu, malesuada ac, dui. Mauris nibh leo, facilisis non, adipiscing quis, ultrices a, dui.

Morbi luctus, wisi viverra faucibus pretium, nibh est placerat odio, nec commodo wisi enim eget quam. Quisque libero justo, consectetuer a, feugiat vitae, porttitor eu, libero. Suspendisse sed mauris vitae elit sollicitudin malesuada. Maecenas ultricies eros sit amet ante. Ut venenatis velit. Maecenas sed mi eget dui varius euismod. Phasellus aliquet volutpat odio. Vestibulum ante ipsum primis in faucibus orci luctus et ultrices posuere cubilia Curae; Pellentesque sit amet pede ac sem eleifend consectetuer. Nullam elementum, urna vel imperdiet sodales, elit ipsum pharetra ligula, ac pretium ante justo a nulla. Curabitur tristique arcu eu metus. Vestibulum lectus. Proin mauris. Proin eu nunc eu urna hendrerit faucibus. Aliquam auctor, pede consequat laoreet varius, eros tellus scelerisque quam, pellentesque hendrerit ipsum dolor sed augue. Nulla nec lacus.

Suspendisse vitae elit. Aliquam arcu neque, ornare in, ullamcorper quis, commodo eu, libero. Fusce sagittis erat at erat tristique mollis. Maecenas sapien libero, molestie et, lobortis in, sodales eget, dui. Morbi ultrices rutrum lorem. Nam elementum ullamcorper leo. Morbi dui. Aliquam sagittis. Nunc placerat. Pellentesque tristique sodales est. Maecenas imperdiet lacinia velit. Cras non urna. Morbi eros pede, suscipit ac, varius vel, egestas non, eros. Praesent male-

Table A.1 A table in the appendix.

System	Author
TEX	Donald Knuth
LATEX	Leslie Lamport

suada, diam id pretium elementum, eros sem dictum tortor, vel consectetuer odio sem sed wisi.

A.2   A Second Section

Etiam ac leo a risus tristique nonummy. Donec dignissim tincidunt nulla. Vestibulum rhoncus molestie odio. Sed lobortis, justo et pretium lobortis, mauris turpis condimentum augue, nec ultricies nibh arcu pretium enim. Nunc purus neque, placerat id, imperdiet sed, pellentesque nec, nisl. Vestibulum imperdiet neque non sem accumsan laoreet. In hac habitasse platea dictumst. Etiam condimentum facilisis libero. Suspendisse in elit quis nisl aliquam dapibus. Pellentesque auctor sapien. Sed egestas sapien nec lectus. Pellentesque vel dui vel neque bibendum viverra. Aliquam porttitor nisl nec pede. Proin mattis libero vel turpis. Donec rutrum mauris et libero. Proin euismod porta felis. Nam lobortis, metus quis elementum commodo, nunc lectus elementum mauris, eget vulputate ligula tellus eu neque. Vivamus eu dolor.

Nulla in ipsum. Praesent eros nulla, congue vitae, euismod ut, commodo a, wisi. Pellentesque habitant morbi tristique senectus et netus et malesuada fames ac turpis egestas. Aenean nonummy magna non leo. Sed felis erat, ullamcorper in, dictum non, ultricies ut, lectus. Proin vel arcu a odio lobortis euismod. Vestibulum ante ipsum primis in faucibus orci luctus et ultrices posuere cubilia Curae; Proin ut est. Aliquam odio. Pellentesque massa turpis, cursus eu, euismod nec, tempor congue, nulla. Duis viverra gravida mauris. Cras tincidunt. Curabitur eros ligula, varius ut, pulvinar in, cursus faucibus, augue.